JOURNAL
OF THE
AMERICAN WATER WORKS ASSOCIATION

VOL. 25 JANUARY, 1933 No. 1

CONTENTS

***All correspondence relating to the publication of papers* should be addressed to the editor, Abel Wolman, 2411 North Charles Street, Baltimore, Maryland.**

"Owing to the present financial difficulties in maintaining an adequate budget, no reprints of articles will be furnished to contributors free of cost. Reprints may be purchased at the usual prices, which will be furnished with galley proof."


"WATCH THE WHEELS GO 'ROUND"
Watching the wheels go 'round used to be a fascinating boyhood pastime. For most of us it's now an obligation and any sign that the wheels are not turning steadily means "get busy".
It's easy to "watch the wheels go 'round" in W&T Vacuum Chlorinators because the working parts are under the glass bell jar, always in plain sight. Don't play "hide and seek" with trouble—know what's happening by installing W&T Visible Control Vacuum Chlorinators on your plant.
Technical Publication No. 38—a copy free—tells all about them.
"THE ONLY SAFE WATER IS A STERILIZED WATER"
WALLACE & TIERNAN, CO.
INCORPORATED
Manufacturers of Chlorine and Ammonia Control Apparatus
Newark, New Jersey
Branches in Principal Cities
Main Factory—Belleville, New Jersey

SA-51

JOURNAL

OF THE

AMERICAN WATER WORKS ASSOCIATION

The Association is not responsible, as a body, for the facts and opinions advanced in any of the papers or discussions published in its proceedings
Discussion of all papers is invited

Vol. 25 JANUARY, 1933 No. 1

TYPHOID FEVER EPIDEMICS FROM WATER SUPPLY IN GERMANY[1]

By Hayo Bruns

(Institute for Hygiene and Bacteriology, Gelsenkirchen, Germany)

The interesting work of Abel Wolman and Arthur Gorman, "The Significance of Water-Borne Typhoid Fever Outbreaks, 1920–1930"[2] moves me to publish some observations which we made in Germany in a similar situation. We can draw valuable lessons from the American work and I hope that the American water-works engineers will also have some interest in our experience.

In Germany typhoid fever has been continually decreasing for a number of years. The incidence and the mortality of the disease in the state of Prussia between the years 1913–1930, is given in table 1. Until 1925, the typhoid and para-typhoid cases are not distinguished from one another, but since the year 1926 this distinction is accurately made.

In no year since 1913 has the number of typhoid cases in Prussia been so small as in 1930, which showed a reduction below 1929 of over 20 percent. This decrease is traceable, not alone to the improvement of drinking-water conditions, but primarily it is the result of the systematic "campaign" against typhoid which, instigated by Robert Koch, has been carried on all over Germany since about the

[1] Translated by E. M. H.
[2] Journal, February, 1931, page 160.

year 1901. It includes greater attention to reporting typhoid cases, thorough care of every patient, isolation of carriers, innumerable bacteriological investigations of the environment and of the patient;[3] inquiry into the cause of every case and thorough disinfection measures.

Nevertheless, of course, providing a faultless drinking water supply has become the most important part in preventing the occurrence of and death from typhoid. I can speak with greater safety concerning

TABLE 1

Typhoid fever in Prussia

YEAR	CASES		DEATHS	
	Total	Per million	Total	Per million
1913	10,011	246	1,022	25
1914	14,524	352	1,375	33
1915	17,256	420	1,515	36
1916	11,680	284	976	24
1917	23,193	570	1,972	48
1918	19,164	478	2,123	53
1919	19,569	528	1,819	49
1920	16,952	440	1,470	38
1921	15,810	426	1,224	33
1922	9,365	244	881	23
1923	12,021	310	999	26
1924	13,086	336	1,068	27
1925	9,751	256	899	24
1926	10,440	274	942	25
1927	5,927	155	546	14
1928	5,334	137	558	14
1929	4,673	119	442	11
1930	3,624	92	368	9

that district which is directly supervised by me—namely the Ruhr coal district—where, certainly, as is doubtless also realized in America, the maintenance of the water supply is of extraordinarily great importance. It meets, however, with very grave difficulties. Since the great typhoid epidemic which took place in Gelsenkirchen in 1902, the improvement of the water supply is the essential cause for

[3] In the institute conducted by me, there are, nearly every year, about 30,000 typhoid investigations carried on, while the number of typhoid cases occurring in our "working area" amounted only to about 310.

the decrease of typhoid. Although, for a period of years in our past, we were pictured as the typhoid breeding-pot of the Ruhr coal section, notorious for its high incidence of typhoid fever, now we are in a more favorable position than the lowest corresponding Prussian figures[4] would indicate. This was accomplished by the addition of chlorine to the drinking water in 1910, and thereafter as it was necessary.

Messrs Wolman and Gorman, in their article already referred to, have treated "drinking-water-typhoid-epidemics" from the technical standpoint. I desire, in this supplement, to emphasize the facts upon which the physician and hygienist base the connection between contamination of drinking water and typhoid epidemics. In my opinion, the epidemiologists are continuously being forced by the laity, to determine the exact cause of every individual outbreak of typhoid and are remiss in their duty if they fail to do this.

The outbreak of a typhoid epidemic is through a complexity of events which are difficult to separate into their causes and effects, so that, in general, one can give a verdict, not with absolute certainty, but only with more or less accuracy. In order to settle in the human intestine, the typhoid bacilli must be carried into the body through the mouth. We must concede, therefore, that for an epidemic to establish itself, the bacilli-infected foodstuffs must accidentally enter a great number of people at the same time. And of those people who eat these foodstuffs in sufficient quantity a number will almost simultaneously become ill with typhoid. In all likelihood, the degree of the infection from this "common" food stuff will differ according to the virulence of the bacilli, or according to the physical conditions of the afflicted persons; or according to the season; and certainly according to the amount of the "food" which each individual eats and according to the "content of bacteriophage" of those persons who are simultaneously made ill by the eating of this foodstuff. This kind of "foodstuff" can equally often be water or milk or milk products; seldom mineral water, salads, vegetables. With other foods one assumes, for the most part, a direct contamination through some "carrier" or typhoid patient; in the case of water, the cause of the

[4] In the years 1929 and 1930 there were in Prussia, out of each one million population, 119 cases in 1929 and 92 cases in 1930; of these, 11 died in 1929 and 9 in 1930. In the Ruhr section, on the other hand, out of each 1 million population, there were only 82 cases in 1929 and 74 in 1930; of these, 8 died in 1929 and 7 in 1930.

epidemic can be the entrance of the typhoid bacilli from toilets, from sewers into the various parts of the water system.

Typhoid epidemics, known to have been caused by the contamination of pipelines, have, in general, very definite characteristics. Before anything else, the area in which the cases of epidemic occur must be circumscribed. This is done through the sanitary commissioners' inquiry into every case (in Germany each case is in the care of the district medical officer) and, through individual investigations, the possibility of another community foodstuff (such as milk) being the cause must be ruled out. In the case of "drinking-water-typhoid-epidemics," it is further necessary that the area in which the typhoid cases occur be circumscribed in its entirety—along with the area of the contaminated water pipe-lines. In this connection the fact must be considered that, naturally, these two areas will not be one and the same,—that is, the water area and the typhoid area. It always happens that people who live outside the "typhoid field" become ill—people who have drunk the infected water either at their places of work or on a chance visit. In the case of the large typhoid epidemic in Gelsenkirchen in 1901, a large number of miners, (in all about 10 percent of the typhoid cases), living in typhoid-free places, became ill. They, however, worked at the mines which were connected with the pipe-system. Even in the typhoid epidemic in Hanover in 1926, more than 20 percent of those who became ill were living in the outskirts where the infected water did not reach at all or only in part. According to the records, a large number of those infected had employment in the contaminated city section or had visited relatives who lived there.

We believe, further, that because the infection of a pipe system takes place only within a limited period of time, in typical cases, outbreaks of typhoid epidemics occur very suddenly in an explosive-like manner. In the same manner, the number of typhoid cases fall off rather suddenly because the typhoid bacilli again disappear from the pipe system. The decline of the epidemic, however, is not quite so sudden as the onset, due to the fact that the duration of the incubation period varies markedly. In consideration, then, of the incubation time and its variations, the date of origin of the different cases should be exactly the same time. The exact determination of the incubation period—so important for finding out the cause of a typhoid outbreak—is in individual cases so very difficult, that frequently, one can hardly set the time of infection with desirable

accuracy. Moreover, it is not often easy (in addition to determining the typhoid) to state accurately the time of the first symptoms (headache, dizziness, general malaise, gastro-intestinal symptoms, slight

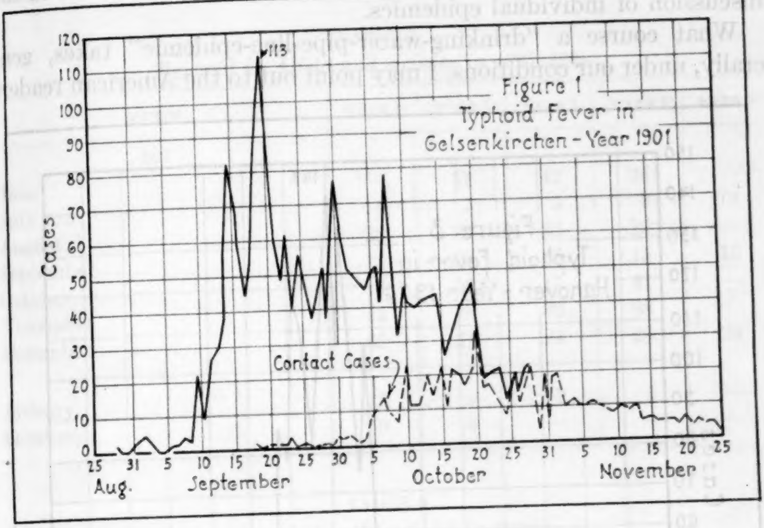

FIG. 1

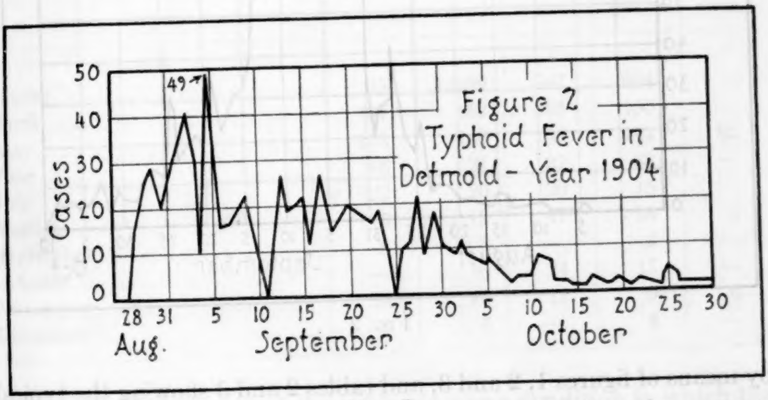

FIG. 2

elevation of temperature, etc.). The answer to the question, whether, considering the variations of the incubation period, all the cases related to an epidemic go back to a single, definitely determined "date," often requires a great deal of critical and intimate knowledge.

But, finally, in order to assume a "causal connection" between the typhoid epidemic and the pipe-lines, a not too small percentage of the persons who have drunk the "suspected water" must become ill. How high these figures are, in general, will be spoken of later in the discussion of individual epidemics.

What course a "drinking-water-pipe-line-epidemic" takes, generally, under our conditions, I may point out to the American readers

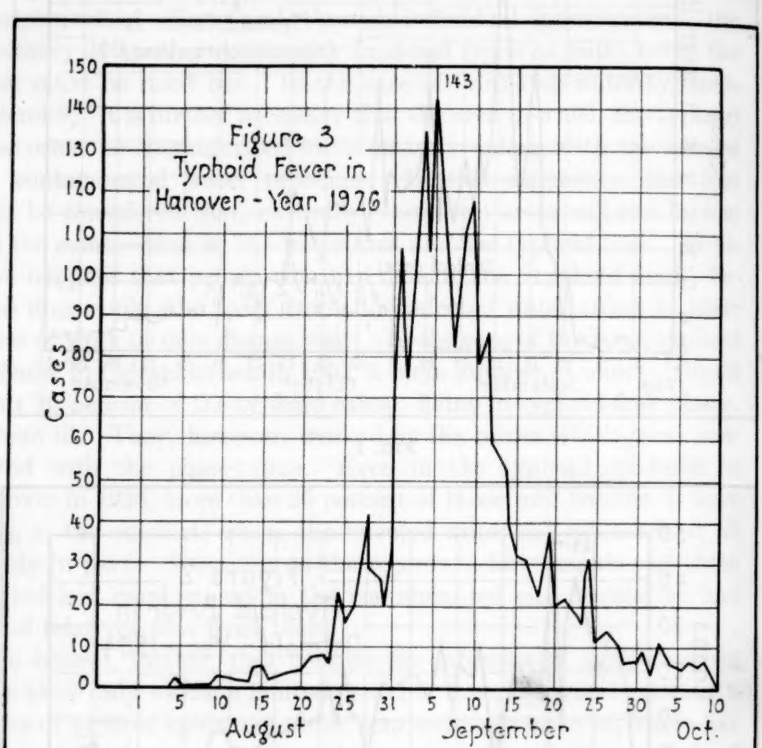

FIG. 3

by means of figures 1, 2 and 3, and tables 2 and 3 showing the typical curves and records of the most widely known German epidemics, namely Gelsenkirchen, Detmold, Hanover, Alfeld and Pforzheim.[5]

[5] It is unfortunate that the charts are not entirely comparable, since some are based upon the daily and others on the weekly records; in the case of the Gelsenkirchen epidemic the charts refer to the number of typhoid cases on the day of "reporting" the illness; in the Hanover epidemic the number of cases are recorded according to the date of the "beginning" of the illness.

In the case of the Gelsenkirchen epidemic of 1901, the opinion of the majority of experts there was that it was caused by the penetration of unfiltered Ruhr water, which water had been infected through the "waste pipes" from the home of a typhoid patient living not more than 300 meters above the Ruhr. This water got into the water-lines.

TABLE 2

Typhoid in Alfeld and vicinity in 1923–1924

MONTH	WEEK 1	WEEK 2	WEEK 3	WEEK 4	WEEK 5
1923					
June	—	37	42	24	—
July	14	7	5	6	15
August	23	75	32	22	
September	15	23	25	15	15
October	13	56	17	41	—
November	43	48	89	68	—
December	69	31	24	24	24
1924					
January	13	9	9	4	—
February	9	4			

TABLE 3

Typhoid in Pforzheim in 1919

MONTH	WEEK 1	WEEK 2	WEEK 3	WEEK 4	WEEK 5
1919					
March	15	1,059	597	524	
April	197	182	224	150	—
May	153	140	133	143	82
June	46	21	32	38	—
July	22	20	20	15	—
August	5	12	41	30	3
September	6	3	—	3	—
October	—	16	24	15	—
November	26	16	11	9	6
December	1	6	—	4	—

In Pforzheim the cause of the epidemic was a condition in which the dejecta of a typhoid patient found their way from the house of the patient, over a (frozen then thawed) slope into a meadow where the wells of the city water-works were located. In Hanover, the greatest probability was the following: the drain water from a dirty gutter

(in an unsewered section of the city) in which there was a great deal of the forbidden overflow from outhouses (or toilet tanks), ran directly into a small brook. Below the brook, protected only by an inadequate layer of gravel of about 80 centimeters, lay the infiltration gallery of the water-works. It is said, in reference to the epidemics of Alfeld and Detmold, that the place where the penetration of the typhoid bacilli into the pipes occurred was not found out with sufficient certainty. If one looks at the curves and tables, he will see that after a short early stage a relatively sharp rise follows in the number of cases reported. After that comes the proportionally shorter "peak" of the epidemic and then a sharper decline of the curve, which is, of course, somewhat more gradual than the rise. After two or four weeks, one or two smaller flareups follow which point to the appearance of so-called "contact cases"—by which is meant those cases where, without doubt, there was a direct infection from one person to another.

It seems to me to be of especial importance, particularly to the water works engineer, to point out that, besides these typical "water-typhoid-epidemics" a series of atypical epidemics have also been observed, which, from the entire accompanying circumstances, could be traced back, with great certainty, to water supply.

Since, in the German literature, and so far as I know, in the foreign literature, little is known of this kind of observation, and since the knowledge of this kind of possibility may stimulate the water engineers to more "active" measures, I am taking the liberty of giving here, in somewhat more detail, some pertinent observations which I have made in the Ruhr coal district, in these last years since the war. One epidemic is of interest, which required in the beginning the overcoming of extreme obstacles to afford an explanation. We were able, however, (after supplementary clarification) to trace it back to the water mains with the kind of assurance that only could be hoped for with epidemiological investigations.

In the City of E, with approximately 500,000 people, unfavorable bacteriological examinations were obtained, from March 7 until October 21, 1919, while in the wells of the only pump station that the city maintained, all the examinations made during this time, always showed favorable bacteriological counts. For unusual reasons (Communist unrest following the war, strikes, the hindrance to commerce etc.) it was impossible to find the cause. At last, it dawned on us that a leak existed between a condensation line, which carried

raw Ruhr water and which had been recommissioned the beginning of March, and the drinking water mains. Consequently, raw Ruhr water, to the extent of approximately 1 to 3 percent of the whole water supply came into the city mains. On October 21, 1919, the leaky gate valve was replaced by an absolutely tight blind flange. Directly after that day, the bacterial count in the city again became normal. By the coexistence of both events, namely the high bacterial count since March 7 and the recommissioning of the condensation pipe at the same time; and the cessation of the influx of Ruhr water on October 21, and the lowering of the bacterial count the same date, was determined, irrefutably, the connection between the pollution of the drinking water and the high bacterial count.

TABLE 4

Typhoid in City X in 1919

	WEEK 1	WEEK 2	WEEK 3	WEEK 4	WEEK 5
January	3	2	0	6	5
February	0	4	4	3	—
March	0	6	0	2	—
April	12	11	0	13	9
May	10	38	15	7	—
June	10	6	3	19	—
July	11	12	5	5	2
August	5	4	12	3	—
September	2	12	0	12	—
October	5	7	0	13	5
November	3	2	3	0	—
December	8	5	0	3	—

Further, it is of interest, that in the communities, St. and K., situated immediately above the water-works of the City X—, whose waste pipes drained into the Ruhr only about 400 meters above the intake pipe of the condensation system—altogether 19 cases of typhoid were reported. Besides, there were several more communities—situated above, whose waste pipes drained into the Ruhr—where typhoid cases occurred to the extent of G, B, and H, one each; Li. 5; St. and Bl. 3 each; La. 29; H. 4; and B. 9. During the first half of 1919, also the Ruhr was abundantly infected with typhoid bacilli. We believe that the typhoid bacilli did not come into the water pipes of the city at one time only but probably entered, in frequently repeated, small doses through the above-mentioned leak. In table

4, the increase of typhoid cases in the City X from April until October is shown.

During the first quarter of 1919, an average of 12 cases a month occurred in the City X—which at that time had 480,000 inhabitants; in April, 1919 there were 45 cases; in May, 70; in June, 38; in July, 35; August, 24; and September, 26; October, 30. In November and December the number dropped down to 8 and 16 respectively. In those seven months in question, there occurred, in all 268 cases; in the remaining five months, 59 cases. It is significant that the district health officer of the City X, by investigating, found a possibility of contact cases in the typhoid occurring up to March and from November on—whereas during the time of the epidemic (April through October) only in the fewest cases was this possible. Since the pollution of the water mains was determined for the first time in October, 1919 and since the bacterial investigations at the pump station was very favorable, the district doctor's suspicion that the water played no part in the epidemic, but that other things such as milk, meat, vegetables, fruit and other common foodstuffs were the cause was certainly unfounded.

It was further suspected, for a little time during the summer of 1919, that the eating of ice cream sold by itinerant street vendors in the City X might possibly be the cause of the typhoid cases. But after the connection between the raw Ruhr water and the water mains became known, even the district health officer accepted, beyond doubt, the fact that the epidemic was caused by the drinking water. Out of all of the 268 cases, about 160 to 180 were directly traceable to the drinking water. Since the City X had about a half million people, this amounted to only 0.3 to 0.4 percent of the population. We attribute this low percentage of typhoid cases to the low percentage (1 to 3 percent) of Ruhr water in the entire drinking water supply.

We have further reasons to believe that the epidemic really was caused by drinking water. From the pumpstation two mains lead into the City E, alternately filling two reservoirs. One reservoir is on St. Hill, the other in B. One of the watermains was connected with the leaky condensation line. This water main which contained 1 to 3 percent raw Ruhr water led to the St. Hill. At that time a high bacterial count and an increase of typhoid cases was reported. These cases occurred not only in certain streets or houses, but were distributed all over the section of the town that was fed from the reservoir on St. Hill. Before we knew the cause of the epidemic—in order

to make an experiment—we switched the watermain which contained the raw Ruhr water to the reservoir B. Immediately, the high bacterial count occurred and three weeks later, the same percentage of typhoid cases in this section E fed from reservoir B. These facts make it unlikely that the condition of ground and soil has anything to do with the origin of typhoid, as maintained by the followers of Pettenkofer's theory. The City E is not level and, like all the country north of the Ruhr, is slightly rolling, but we observed no increase in the number of cases in the lowlands. We did observe that more of the poorer people were affected than people in the better financial classes. In April and May, 1919 there was rarely more than one case in a family. In July, August and September the district doctor reported several contact cases but only relatively few. The few new cases reported in September and October were, in my opinion, caused by a slight sporadic infection of the pipe system. This small epidemic was very interesting and instructive to me because it was in direct contrast to the text-books and the opinion of experts that a drinking-water-typhoid-epidemic begins and stops suddenly (explosively). Of course, immediately after the cause of the epidemic was cleared up, all precautions were taken to prevent any possible further pollution of the water lines.

The absolute number of typhoid cases caused by water-main epidemics is very variable. In the large typhoid epidemics of Gelsenkirchen, Oberschlesien, Mulheim, Pforzheim and Hanover, thousands of people became ill of typhoid; in other epidemics, as in Detmold, Alfeld, Stralsund, hundreds became ill. In the Ruhr district, we have also observed several epidemics where only 18 to 25 people were affected, but which, in consideration of the accompanying circumstances, we had to trace back to the water. On the average, it can be said, that in a typhoid epidemic, the number of infections caused by the polluted water mains amounts to about 0.5 to 4 percent of the population supplied by the infected water.

In the case of the Gelsenkirchen epidemic, in a district (supplied by the infected mains) of some 500,000 people, about 5500 or 1.1 percent became ill. In Pforzheim, however, of about 79,000 inhabitants, 3692 became ill, which amounted to 4.7 percent. In the Hanover epidemic, of the approximately 200,000 persons supplied by the water (only half the city was supplied by the suspected pumpstation) there were 2780 typhoid cases,—1.4 percent. In Detmold, of about 13,000 people, 780 or 6 percent were ill; in Alfeld and its environs (26 vil-

lages) consisting of about 20,000 people, 940 or 4.7 percent became ill. But occasionally, this percentage will go much beyond those mentioned. The relatively worst typhoid epidemic occasioned by drinking water which I personally experienced, broke out in 1920 in the Village K, populated by 700 residents. In this instance, within two or three weeks, more than 140 persons or 20 percent of the people supplied by the water, simultaneously became ill of typhoid. Because the well was seriously infected, and because non-residents who came to the village K on a Sunday during the critical time (and who there drank nothing but water) became ill, one must accept as the cause, the contamination of the water. More interesting, but often more difficult to explain, are the cases in which only a relatively small proportion of the population, supplied with the water, become ill with typhoid. In the City Y, in 1923, we had an increase of typhoid to 40 or 45 cases which meant that out of 220,000 population affected, about 0.2 case occurred per thousand inhabitants. Of course, this was only probably traceable to the water. This was the case: during a sudden, unexpected, summer flood, river water, not sufficiently filtered, got into the "collecting wells." The result of the bacteriological investigations gave definite proof of the pollution of the water which was still further confirmed by various accompanying conditions, for example, the simultaneous outbreak of typhoid cases in several outlying communities supplied by the same pump-station. Another time, we observed that in the City S, populated about 15,000, within a period of 4 weeks 62 typhoid cases occurred. Lacking any other cause (common food—such as milk or other things was not in question) and in view of the unfavorable results of the water examinations and the proximity of a cesspool to the water-works, this could, with probability, be traced to the infection of the water mains with typhoid bacilli. It was significant that about 4 in a thousand population became ill of typhoid. In the winter of 1924 we had in the City Ste. and the Community Sto., (approximately 50,000 inhabitants) both of which were supplied from the same waterworks, about 30–35 typhoid cases. The number of ill people amounted to something under 1 to 1000. Likewise, we had to diagnose this as a water epidemic because of a number of circumstances, namely: the inundation of the waterworks territory; the absence of any other clear-cut possibility; high bacterial counts; lower "Colititer;" also the fact that in the rural community Sto. the disease occurred only in cases where the people were supplied by the water lines.

Also during the "New Year's Eve" flood about the end of the year 1925–26 in a city of 130,000 people, a very small increase in typhoid of about one dozen cases was reported, which we traced to a slight pollution of the drinking water. All the other pumpstations of the Ruhr district added chlorine to the drinking water on account of the floods; the districts supplied by these remained entirely free from typhoid. This particular pumpstation supplied only a part of the City N. Formerly it was not necessary to chlorinate the water because the ordinary floods had scarcely made themselves noticeable around this pumpstation. When this one, alone, neglected to apply chlorine, it was taken unawares by the flood. In the section of the City N. supplied by this station, very promptly, in the third week of January, 12 to 15 cases of typhoid appeared which had to be traced with certainty to the water. Naturally this experience occasioned the building of a chlorination plant there too.

In the City H, with a population of about 12,000, there broke out, last year, within 14 days, a typhoid epidemic of about 70 cases. The city was provided with water from two sources—the pumpstations being located far apart. The water from one pumpstation was perfect; the cases of typhoid occurred entirely in the district supplied by the other, whose well-area was located immediately next to a sewer ditch, carrying waste matter. Very high bacterial counts, and unfavorable "Coli" findings pointed to the water. The results of the investigations were the occasion for the immediate establishment of a chlorination plant and three days before the outbreak of the epidemic, this was completed. It was the general opinion that, if the plant had been built three weeks earlier, the typhoid epidemic might have been avoided. In the subsequent investigations, it developed that the drain water of a house, in which a typhoid carrier lived, flowed into the "sewer ditch." It is especially interesting that the "carrier" had moved here only recently.

It is further laid down by the "text-books" that to concede a "drinking-water-epidemic" the whole area in which the typhoid extends should be the same as the area of the water supply. But on this point there are abundant exceptions. The water will be drunk, for example, at places of work, or on occasional visits, by people outside the infected water area and who become ill outside the infected water area. I know of two typhoid cases occurring in the Ruhr district in the Autumn of 1926. Both patients had paid visits in Hanover during the epidemic there and, it must be admitted, brought back the infection with them.

In explaining the Gelsenkirchen epidemic of 1901, this question played a real part. The miners who, during the day, had the opportunity to drink the infected water at the mines, became ill in communities which did not belong to the district supplied by the water-works and where otherwise no typhoid developed. When the people of a city are supplied with water from two stations which are under the same management, it is quite natural that there is a connection between the water mains of the several systems. In such a case it is impossible to ascertain just how far the polluted water of one system will penetrate into the other. Due to uneven consumption and varying pressure in the different systems, the water will, more or less, flow back and forth from one system to another. One would have to concentrate particularly on this point if he is to obtain an answer to the question whether the distribution of the typhoid cases within an individual city section can be explained by the contamination of a single station. In Hanover, the situation was this: According to the fixed limits of each of the districts to be provided by the three pumpstations, about 70 percent of the first cases of typhoid were located within the district supplied by the "accused" pumpstation. But, if one accepts the further limits, due to this backward and forward movement of the water, then more than 90 percent of the cases were within this district. Besides, it was estimated, that of the typhoid patients located outside this district, about two-thirds (in the beginning of the epidemic, an even larger part) had got the infected water, either at their places of work or at points of contact within that section of the city affected. A number of outside districts, which certainly did not receive the infected water, proved to be entirely free of typhoid—with the exception of a single case which occurred there but which was well established as a contact case.

Further definite differences between the "waterfield" and the "typhoidfield" may occur from the fact that, perhaps only a part of the pumpstation is polluted with typhoid bacilli and the districts dependent upon the water may not be supplied by all the units of the water-works at the same time, but individual parts of the district may correspond to separate parts of the water-works. This condition once played a part in the Gelsenkirchen epidemic of 1901, inasmuch as portions of the district involved, but which were supplied with water from a different high reservoir, remained typhoid-free. You will recall that the question was discussed previously, whether the water to which the epidemic had been traced, was flowing by means

of a certain gate valve into the contaminated or uncontaminated district. This question came under lively discussion again on the occasion of a typhoid epidemic which, in May, 1911 broke out in a number of communities which were supplied by the same waterworks. One of these communities, the one situated farthest away from the water supply—the City Do., remained typhoid-free. On the basis that it was a small community of only 7000, this immunity could not be explained. It was to be expected that, since, Do. was located in the same provision-area where 1500 of the 300,000 inhabitants (0.5 percent) became ill, cases would have occurred in Do. also. Upon making examinations, it was found that the water in Do. came through a special pipe-line from a certain "unit" of the waterworks not contaminated by typhoid bacilli.

TABLE 5

Number of typhoid cases in City N in 1925

	WEEK 1	WEEK 2	WEEK 3	WEEK 4	WEEK 5
May	—	1	1	0	3
June	0	9	8	0	—
July	0	3	3	0	—
August	0	0	3	1	3
September	2	0	1	3	—
October	89	82	31	29	—
November	19	14	2	2	0
December	0	0	1	0	

A similar, apparent incongruity between the "waterfield" and the "typhoidfield" seemed to be exhibited in the beginning of a typhoid epidemic which broke out in a Village N. in 1926. In N., about 30 typhoid cases occurred in July, 1926. Immediately the water supply was blamed for it. But since the typhoid cases had appeared, for the most part, in the inner section of the city, the idea was not further pursued; at all events, nothing was done about it. Then suddenly, at the beginning of October about 250 new cases occurred in the city proper, within a period of 8 days, which stimulated a renewed effort to find the origin of the epidemic. The record is shown in table 5. Hitherto, deficient sewerage, unsatisfactory soil conditions, contact infections, food infections, etc., had been suspected as the cause, but these conjectures were not able to explain satisfactorily the sudden renewed flareup of typhoid.

This was the situation: the Village N. was supplied with water from three sources. The one seemed, upon outward inspection, to be satisfactory; the second was only a little suspicious; but the third was situated rather unfavorably. This third, thoroughly unhygienic station supplied the water, first of all, to the center of the city where the typhoid cases had occurred. Six wells lay parallel to an open brook only 1 to 2 meters away from them. Supposedly, of course, there should be no connection between the wells and the brook. But by damming up the brook and by using fluorescein, we could determine that there was a connection. A quarter of an hour after applying the fluorescein to the brook, in one of these wells, and shortly thereafter, in the collecting well of this supply, the water showed an intensely green color. By further following up the brook, it was found that, in rainy weather, the brook received the overflow from a manure-trough from a farm house located about 1 to $1\frac{1}{2}$ km. above it. Into the manure-trough, also went the fecal matter from the house. An old man living in this house told us that, in the last half year, he had been ill for a long time with an undiagnosed fever but had not called a physician. Although, upon subsequent examination he was not found to be a "carrier," nevertheless, it seemed to us that the circumstantial evidence was so conclusive as to establish pretty definitely the connection between the typhoid epidemic and the contamination of the wells through this passing brook. Therefore, on the grounds of this probability, steps could be taken to combat the epidemic. Since the pumpstation in question could not be shut off from the system, the first step was to chlorinate the water. Whether the cessation of the epidemic, which followed in the middle of November was influenced by this, must, of course, remain a question. In any case, no further cases developed, despite the fact that other conditions were not appreciably altered (although work on the sewerage system is now being carried on).

I must still speak of another matter in this connection, namely, in what season of the year typhoid epidemics are especially prevalent. Although it is generally believed that typhoid epidemics, whether water-borne or from other causes, only are observed in Summer and Autumn, this is not correct. The great Pforzheim epidemic occurred in the winter and also several others of which I have spoken, occurred in Winter or in Spring. It is correct, only that each Fall, an increase of typhoid cases occurs which, naturally frequently takes on epidemic proportions. It is also correct that the greatest number of typhoid

epidemics occur in the height of the summer. It is interesting in this connection, that the two typhoid epidemics of especial significance, which have occurred in Germany in the last few decades, occurred 25 years apart almost to the day. The typhoid epidemic in Gelsenkirchen broke out the beginning of Sept., 1901; the one in Hanover, the beginning of Sept., 1926. For the belief that, in Autumn, the typhoid virulence is specially great and that also at this time "bacteriophage" is only sparsely present, we really have no scientific basis. Provisional also is the belief that the human body has a greater disposition to typhoid in Mid-Summer and Fall. It is my opinion, that upon this question, we have not reached the end of our knowledge and it is very likely that further observations and investigations can give us much light on this subject, which will perhaps be of great importance in explaining the origin of typhoid epidemics.

All the characteristics, therefore, of "water-borne-typhoid epidemics" follow the mechanism of the conditions of the contamination. I come again to the work of Messrs. Wolman and Gorman referred to in the beginning of this article. It seems to me that from this whole discussion, the conclusion must be drawn that the development of the origin of typhoid epidemics, supposed to be caused by the contamination of water, requires the fullest coöperation between the technician and the medical epidemiologist.

DISCUSSION

Abel Wolman:[6] It is gratifying to note the detailed comment which Dr. Bruns makes on the water borne typhoid fever in Germany, particularly in view of the fact that it confirms the situations pointed out by Gorman and myself with reference to the United States.[7] The problem of continuous and careful control of water treatment to prevent typhoid fever is a universal one. Even while reviewing the manuscript by Dr. Bruns, a water borne epidemic of typhoid fever of considerable size was reported from England. Its importance confirms the admonitions in our earlier contribution.

The outbreak of typhoid fever in the Ecclefechan special water

[6] Editor-in-Chief, The Journal of the American Water Works Association; Chief Engineer, Maryland Department of Health, Baltimore, Md.

[7] The Significance of Water-Borne Typhoid Fever Outbreaks. Wolman and Gorman. The Williams & Wilkins Company, 1931.

district[8] of England affected 55 per 1000 of the inhabitants (primary cases only). Of a population of about 630, 64 cases in total were reported. The evidence indicated the public water supply as the vehicle of infection and a typhoid carrier was found employed in the water collecting area. The water supply was from wells and springs. Examination of the collecting pipes showed that surface pollution gained access to the lines conveying the spring water to the collecting tank, the epidemic thus falling within classification "G," as suggested by Wolman and Gorman. Vigilance still remains the key word to water works treatment control. In these days of budget slashing, these words should be posted where every taxpayer can see them.

[8] A Water-Borne Epidemic of Typhoid Fever. Ritchie and Armstrong. The Journal of Hygiene, 32: 3, July, 1932, page 417.

HYDRAULICS OF RAPID FILTER SAND

By Roberts Hulbert and Douglas Feben

(Senior Chemists, Filtration Plant, Detroit, Mich.)

The present-day rapid sand filter performs reasonably well, but perhaps not markedly better than it has for years past. Actually, we do not know as much concerning the hydraulics of the stratified, rapid-sand filter bed, as of the old slow-sand filter, which Allen Hazen made the subject of his noteworthy and far reaching studies. How and why the layer of graded sand in a modern rapid filter functions under the various and variable conditions that it does, still remains somewhat of a mystery.

Safe precedent still dictates a restricted choice in the selection of filtering material, as to kind, depth, size, shape, grading, and what not; yet controversy over these fundamentals suggests that while such precedent may be safe, it may also prove both costly and unhappy.

At present, numerous and widespread studies are concerned with the task of obtaining some basis of fact as a guide to sand bed design, and its effect on filter performance. The outcome may possibly be that all precedent will be brushed aside. At least a more rational basis for the specification of filtering materials may be expected as a result, something which is badly needed for sake of progress, if nothing more.

Notably the question of filter clogging rate under different conditions of sand bed composition and applied water is now receiving much attention. This problem is a very complex one, involving, as it does, a multiplicity of variables with unknown interrelationships. True interpretation and correlation of the voluminous facts and data from every-day filter plant performance are difficult, if not in many ways impossible, and the same is apt to be true of the most carefully controlled experimental filter studies involving the filtration of coagulated water. One apparent reason is that too little is known of the pure hydraulics of the flow of clear water through a clean, stratified layer of sand.

The present study deals solely with the important, fundamental

factors known to govern initial filter loss-of-head, viz., sand size, depth, porosity, rate-of-flow and water temperature. The interrelationships between these variables have been carefully studied and evaluated. This work is to be considered as preliminary to a more inclusive study of filter rate-of-head-loss, or clogging rate, under actual conditions of filtration; and should, it is believed, assist greatly in the interpretation of these later studies of a more complex nature. However, it is hoped that both designing engineer and plant operator alike may find something of real value in this present study.

DESCRIPTION OF APPARATUS

Since the apparatus used in this study was designed more especially for use in subsequent rate-of-head-loss experiments, it provided some integral features not strictly necessary to the present investigation, such as individual filter rate-of-flow controllers, and provision for the application of two or three different kinds of applied water at one time. Filter tubes larger than the usual $1\frac{3}{4}$-inch experimental type were chosen in order to minimize the rise in water temperature which sometimes occurs during flow through the smaller tubes, with consequent release of dissolved oxygen to create false and unduly high loss-of-head.

These filters are built of 6-inch Pyrex glass tubing, 50 inches long mounted and clamped between two special brass castings, and sealed with rubber gaskets and cement. The lower casting is funnel-shaped and filled with graded gravel to equalize wash-water distribution. The sand layer is entirely separated from the gravel by a perforated brass disc covered with a 100-mesh brass screen, both secured to the top of the lower casting. This screen serves a two-fold purpose: it gives a definite base line from which to measure sand depths, and permits the removal of the entire sand sample, to the last few grains, without admixture with gravel. The upper casting is slightly conical to prevent air trapping, is supplied with an $1\frac{1}{2}$-inch plug to permit the introduction and removal of sand without the necessity for dismantling the tube, and is further supplied with an air cock to release trapped air. In addition the apparatus includes the necessary valves and piping for the influent, effluent and wash-water, loss of head gauges, individual rate-of-flow controllers, and cocks for air release at all points where this may accumulate. A thirty inch depth of sand in these filters has a volume less than one-half cubic foot, a quantity not too large for easy handling.

The empty tubes were first calibrated for loss of head at different rates of flow, and all subsequent loss-of-head readings taken with sand in the filters were reduced to the basis of sand loss only, by subtraction.

EXPERIMENTAL PROCEDURE

The sand sample was poured into the filter tube and thoroughly backwashed. The wash-water valve was then closed very slowly, not only to simulate large-scale plant practice, but more for the reason that this procedure was found to be the only way to obtain a fixed and definite sand depth, one which could be duplicated with successive back-washings. If the sand depth is allowed to vary, the porosity of the sand column varies accordingly, and unless a correction for this is applied, loss-of-head measurements repeated on the same sample, are neither the same nor comparable. After washing and settlement of the sand, the operating head is applied to the filter, and all parts of the apparatus are blown off to eliminate trapped air.

In all these experiments only clear, filtered water was applied to the sand and consequently no clogging of the bed took place. Under such conditions rate-of-flow controllers are unnecessary and were not used. The effluent from the filter units was discharged directly through a filter-to-waste valve, and rates of flow were changed or adjusted by hand operation of this valve.

The reading of sand depth was taken after opening the effluent valve, since a slight initial subsidence of the sand bed takes place when any filter is first put in service after washing. No further reduction in depth was ever noted in these experiments. However, any jarring or vibration of the apparatus would have resulted in further compacting of the sand layer, and because of this was studiously avoided, as a sure source of error in depth measurements, and furthermore as not representative of plant-scale conditions.

The rate of flow for any effluent valve setting was obtained by collecting and measuring the filter discharge for a definite time interval, using a graduated cylinder and stop watch. Having previously calibrated the tubes for cross sectional area, or diameter, the volume of discharge per minute was readily converted into m.g.a.d. rate (million gallons per acre per day) by multiplying by a factor computed for each individual filter tube. Two or more such measurements were made at each set rate-of-flow, and the temperature

read immediately before and after each measurement of discharge. Rate-of-flow measurements were accepted as accurate only when the temperature variation, during a succession of different rate settings, was not more than one degree F., and when successive volume discharge readings for the same rate did not vary more than two percent. Loss-of-head readings were taken simultaneously with those of rate-of-flow, to the nearest 0.01 foot.

The original plan was to carry the experiments through the summer, fall and winter seasons, to cover the entire range of falling water temperatures, but at the conclusion of this series some of the measurements were found to lack sufficient accuracy for definite interpretation, and were discarded. The same series were repeated during the later season of rising water temperatures. The results of the earlier work were by no means a total loss, as the first experience served to develop the best technique, and insure a greater degree of accuracy in these later data.

In attempting to evaluate the effects of a number of causes, collectively responsible for a given phenomenon, it is desirable for the sake of accuracy to isolate one variable at a time, and study its individual effect. If, from preliminary observations, one variable apparently predominates (sand size, for instance, in the present study) it is then advisable to arrange an experimental procedure that will eliminate all others, so that the effects of this important variable can be reduced to a mathematical expression. Repetition of this procedure finally results in an exact formulation which takes all the known variables into account, both individually and collectively.

The loss-of-head resulting from the passage of clear water through clean sand is dependent upon a number of factors, the most obvious of which is the diameter, or size of the sand grains. As it exists in the filter bed the sand is stratified, or graded by the wash-water into a number of consecutive layers, increasing in particle size from top to bottom. The sand comprising any one of these layers is almost uniform in size. As water flows through such a bed, each layer of sand contributes to the total loss-of-head a value dependent upon its particular sand size and depth. By separating each layer of distinct and uniform sand size from those adjoining, the loss-of-head which it contributes may be accurately evaluated. This was accomplished by sieving out eight, separate fractions, from clean Ottawa sand, each of the highest possible degree of uniformity of size, and differing from each other only in the property of size.

DESCRIPTION OF EXPERIMENTAL SANDS

These uniform, round-grain sands, numbered 1 to 8 in order to size, were used as a basis of this study. Figure 1 shows a graphical plotting of them on semi-logarithmic paper. The grading curve for each one is shown as a dashed, straight line, terminating at each end in the calibrated size of the two limiting sieves. Obviously it is not possible to draw more accurate grading curves than those shown, except by the use of other intermediate sieves. Every procurable screen in this Tyler series was employed in the separation of these eight uniform sands.

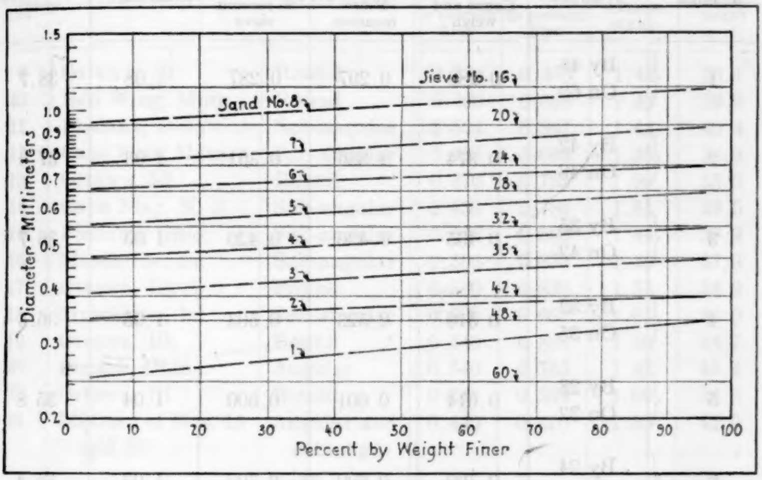

FIG. 1. GRADING CURVES FOR UNIFORM OTTAWA SANDS, NO. 1 TO 8

Size determinations on each of these eight fractions were made by three independent and distinct methods, the results of which are shown in columns 3, 4, and 5 of table 1. The "count and weigh" method (column 3) for determining average particle size is that devised by Allen Hazen, and recommended in "Water Works Practice." When applied to round-grain sands such as these, it undoubtedly gives very accurate results. The "micro-measure" given in column 4 represents the mean microscopic diameter of a large number of representative grains, measured under the microscope using an ocular micrometer. Column 5 gives the 50 percent, or median size by weight, read from the grading curves plotted from a sieve analysis; for example those shown as figure 1. In the case of seven of these

eight uniform sands the "count and weigh," or average-weight-size, is slightly greater than the median 50 percent size determined by sieve separation. This is believed to be due to the effect of the long continued screening which these sands underwent, causing more of the larger grains to pass through a sieve into any one fraction than would

TABLE 1

Characteristics of uniform Ottawa experimental sands

1	2	3	4	5	6	7
SAND NUMBER	SIEVE NUMBERS	MEAN DIAMETER IN MILLIMETERS			UNIFORMITY COEFFICIENT	POROSITY (PERCENT VOID)
		Count and weigh	Micro-measure	50 percent sieve		
1	By 48 On 60	0.289	0.297	0.287	1.08	38.7
2	By 42 On 48	0.374	0.386	0.361	1.03	37.0
3	By 35 On 42	0.435	0.426	0.420	1.05	36.7
4	By 32 On 35	0.510	0.523	0.503	1.05	36.3
5	By 28 On 32	0.614	0.601	0.600	1.04	35.8
6	By 24 On 28	0.709	0.690	0.700	1.03	35.4
7	By 20 On 24	0.899	0.908	0.831	1.06	35.3
8	By 16 On 20	0.997		1.025	1.05	35.2

happen in the case of the usual five-minute sieving period. At the same time, some of the first small grains which fall into this fraction, and which in case of the shorter sieving might be retained therein, would eventually, in the course of long, continued shaking, pass into the next smaller sized fraction. The final result of these two effects is to increase the average grain size of the several fractions, when determined by the method of "count and weigh."

From the size characteristics of these eight uniform sands as given, it is obvious that in any series of experiments performed with any one of them, the factor of sand size is virtually a constant, in all experiments, as well as throughout the entire depth of the experimental filter bed in any single experiment.

TABLE 2

Characteristics of sixteen graded test sands

1	2	3	4	5	6	7
SAND NUMBER	SOURCE	GRAIN SHAPE	SIZE IN MMS. 10 percent finer	60 percent finer	UNIFORMITY COEFFICIENT	POROSITY (PERCENT VOID)
9	Ottawa, Ill.	Round	0.324	0.465	1.43	36.1
10	Red Wing, Minn.	Round	0.490	0.698	1.42	33.9
11	Phalanx, Ohio	Sub-angular	0.621	0.897	1.44	40.4
12	Cape Cod, Mass.	Sub-angular	0.491	0.663	1.35	38.9
13	Ottawa, Ill.	Round	0.370	0.735	1.99	35.0
14	Cape May, N. J.	Sub-angular	0.430	0.780	1.81	35.5
15	Phalanx, Ohio	Sub-angular	0.416	0.600	1.44	39.9
16	Muscatine, Ia.	Sub-angular	0.595	0.870	1.46	37.0
17	Ottawa, Ill.	Round	0.550	0.830	1.51	34.9
18	Muscatine, Ia.	Sub-angular	0.600	0.987	1.65	36.0
19	Ottawa, Ill.	Round	0.540	0.808	1.50	34.7
20	Copley, Ohio	Angular	0.540	0.765	1.42	48.4
21	Ottawa, Ill.	Round	0.341	0.544	1.60	34.5
23	Mixture of Nos. 15 and 20	Angular and sub-angular	0.440	0.670	1.52	42.9
24	Mixture of Nos. 15 and 20	Angular and sub-angular	0.479	0.730	1.52	45.6
25	Mixture of Nos. 15 and 20	Angular and sub-angular	0.441	0.660	1.50	41.7

In addition to these eight uniform sands just described, some sixteen representative, graded filter sands of seven different kinds were studied. The latter all came within the usual filter sand range in respect to grain size, shape and uniformity. Their characteristics are listed in table 2, and the results of the ordinary, laboratory screen separations in table 3. Referring to table 3, the percentages by weight found for the several sieve fractions are tabulated cross-

TABLE 3

Sieve separations of sixteen graded test sands

1	2	3	4	5	6	7	8	9	10	11	12
SAND NUMBER	PERCENT BY WEIGHT OR DEPTH BETWEEN SIEVES										
	10 / 14	14 / 16	16 / 20	20 / 24	24 / 28	28 / 32	32 / 35	35 / 42	42 / 48	48 / 65	65 / 100
9	0	0	0.01	0.09	0.40	8.90	33.70	23.03	20.32	12.21	1.34
10	0.90	1.60	7.00	19.00	22.20	28.05	14.91	3.59	1.62	0.96	0.17
11	9.80	7.80	18.10	27.7	20.0	14.43	1.95	0.06	0.02	0.10	0.04
12	0.50	1.00	4.40	15.90	21.50	31.40	20.14	4.25	0.27	0.60	0.04
13	0	0.60	8.50	29.70	6.60	15.30	18.90	8.07	6.87	4.94	0.52
14	5.90	5.00	14.6	18.2	13.1	12.5	15.8	7.70	4.00	3.16	0.04
15	0	0.10	1.3	12.8	15.3	22.50	29.32	13.17	3.94	1.58	0
16	1.10	4.10	23.30	38.80	17.30	8.70	5.01	0.95	0.20	0.54	0
17	0	0.20	16.00	50.60	8.80	14.66	8.49	0.73	0.15	0.37	0
18	5.50	13.8	33.2	27.4	8.9	3.23	4.55	1.47	0.47	1.48	0
19	0	0.30	13.0	45.3	10.27	19.63	9.95	0.85	0.22	0.48	0
20	0.10	0.20	10.95	33.35	24.8	19.30	9.18	1.52	0.06	0.47	0.07
21	0	0.10	5.40	18.40	4.70	10.90	25.30	14.7	11.06	8.69	0.75
23	0	0.10	5.40	19.5	18.3	22.3	20.70	9.88	2.36	1.40	0.06
24	0	0.30	8.5	27.0	21.6	20.2	14.2	5.63	1.28	1.22	0.07
25	0	0.10	4.50	18.50	18.20	22.80	22.15	10.69	2.16	1.24	0.06
Median 50 percent size......	1.59	1.23	1.02	0.83	0.70	0.60	0.50	0.42	0.36	0.28	0.15

TABLE 4

SIEVE NUMBER	CALIBRATED SIZE OF OPENING	MEDIAN, OR 50 PERCENT SIZE BETWEEN SIEVES
	mm.	*mm.*
10	1.862	1.59
14	1.356	1.23
16	1.132	1.02
20	0.930	0.83
24	0.746	0.70
28	0.655	0.60
32	0.548	0.50
35	0.461	0.42
42	0.383	0.36
48	0.339	0.29
60	0.243	

wise, in horizontal rows; the figures in each bracket represent the fraction retained between the two sieves whose numbers are given at the top of the corresponding vertical column.

Table 4 lists the laboratory calibration sizes of all screens in the set used for this work; also the 50 percent, or median size for each pair of adjoining screens. These median sizes were read from a large-scale graph like figure 1, and represent, in fact, the geometric mean size between sieves, which may be calculated by simply taking the square root of the product of the two limiting sieve size calibrations.

The search for a suitable parameter of sand size, for use in the loss-of-head computations for a graded sand, finally resulted in the selection of this median 50 percent sieve size, as the most simple and practical index. It was therefore used in all computations made on the sixteen graded sands, in which the assumption is made that the percentage by weight of each sieve portion may also, without appreciable error, be considered as the percentage by depth; and further that each sieve portion of a graded sand, retained between two consecutive screens, represents a highly uniform sand, entirely comparable in this respect to the eight uniform Ottawa experimental sands previously described.

EFFECT OF RATE-OF-FLOW AND DEPTH OF SAND

In Ellm's text on water purification may be found the statement that initial filter loss of head is strictly proportional to (1) rate-of-flow, and (2) the depth of the sand bed. These two generally accepted principles were fully confirmed during the course of this study, merely as part of the general procedure. Figure 2 has been drawn from these experimental data to illustrate these two important relations. The upper curve shows the proportionality which exists between rate-of-flow and loss-of-head (within the capillary or viscous range of flow). This curve was drawn from data on Ottawa graded filter sand #9, but is, in fact, only one of more than 100 similar graphs obtained during the course of this study, every one quite equally illustrative of this particular principle. The lower curve shows the relation between head-loss and sand depth, at a constant rate of flow and constant water temperature. It was obtained by dividing the sand sample (Ottawa #9) into eight equal, representative portions with a Jones sampler, and then measuring the loss-of-head, at a constant rate and temperature, after each successive portion, or depth increment had been added and the bed re-washed

and graded. Since any variation in the measured loss-of-head due to differences in the experimental rates or sand depths may be cor-

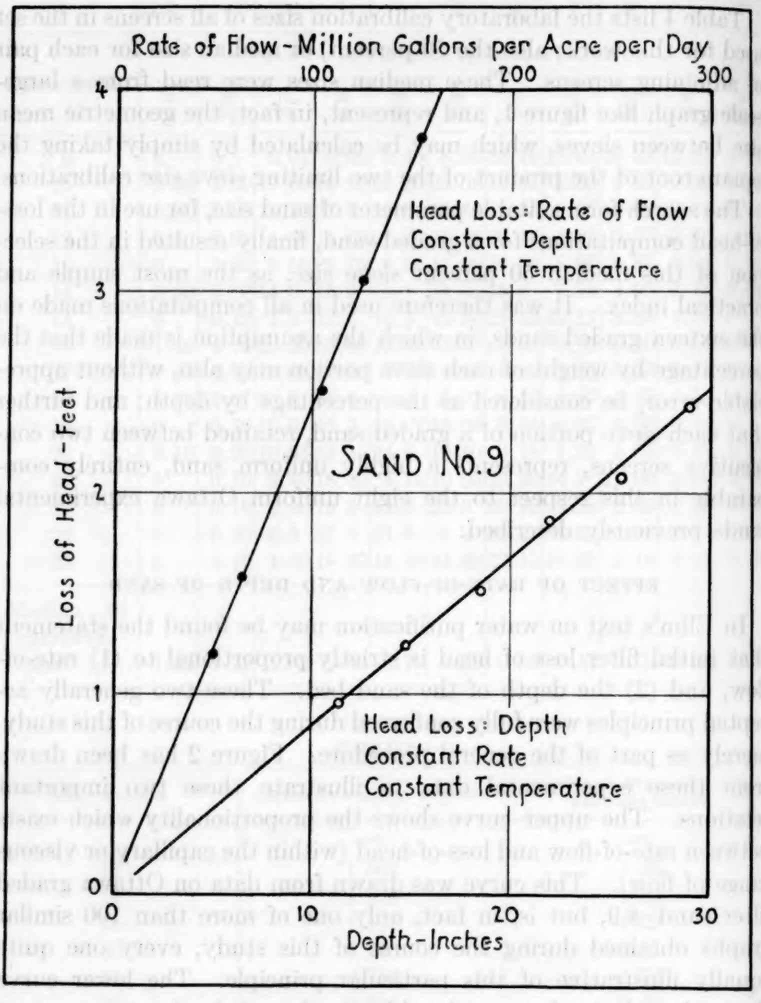

FIG. 2

rected for by direct proportion, all subsequent measurements of head-loss given in this study have been corrected to an arbitrary standard depth of 100 inches, and to the standard rate of 125 m.g.a.d.

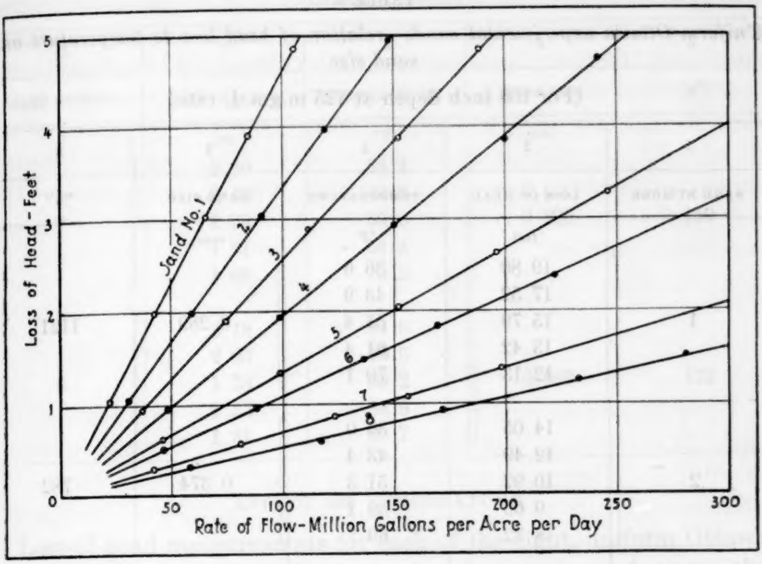
Loss of Head - Feet
Rate of Flow-Million Gallons per Acre per Day
Sand No. 1
2
3
4
5
6
7
8

FIG. 3

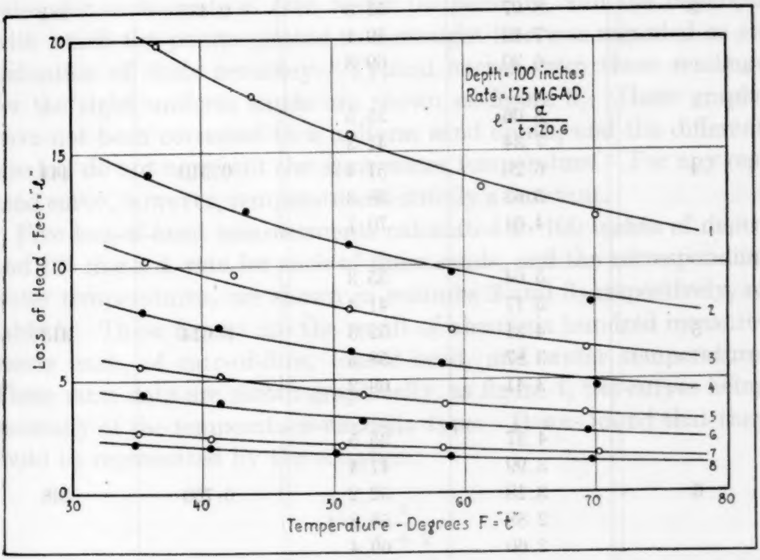
Depth - 100 inches
Rate - 125 M.G.A.D.
$\ell = \frac{a}{t+20.6}$
Loss of Head - Feet "ℓ"
Temperature - Degrees F "t"

FIG. 4

TABLE 5

Uniform Ottawa experimental sands—relation of head loss to temperature and sand size

(For 100 inch depth at 125 m.g.a.d. rate)

1	2	3	4	5
SAND NUMBER	LOSS OF HEAD	TEMPERATURE	SAND SIZE	"a"
	feet	*°F.*	*mm.*	
1	19.80	36.6	0.289	1121
	17.52	43.9		
	15.79	51.4		
	13.42	61.4		
	12.13	70.1		
2	14.05	36.0	0.374	782
	12.49	43.4		
	10.92	51.3		
	9.66	59.1		
	8.32	69.7		
3	10.26	35.7	0.435	581
	9.61	42.5		
	8.07	51.3		
	7.13	59.1		
	6.30	69.3		
4	8.06	35.5	0.510	444
	7.34	41.3		
	6.20	51.4		
	5.63	58.3		
	4.61	70.1		
5	5.64	35.3	0.614	313
	5.17	41.4		
	4.31	52.3		
	3.87	58.5		
	3.41	69.2		
6	4.37	35.5	0.709	238
	3.99	41.4		
	3.18	52.2		
	2.88	58.8		
	2.60	69.4		

TABLE 5—*Concluded*

1	2	3	4	5
SAND NUMBER	LOSS OF HEAD	TEMPERATURE	SAND SIZE	"a"
	feet	*°F.*	*mm.*	
7	2.69	35.1	0.899	150
	2.43	40.6		
	2.08	50.6		
	1.91	58.3		
	1.66	70.2		
8	2.18	34.8	0.997	122
	2.03	40.6		
	1.73	50.2		
	1.53	58.9		
	1.31	69.7		

EFFECT OF TEMPERATURE

Loss-of-head measurements for each of the eight, uniform Ottawa sands were made at five different water temperatures, between the limits 35 to 72 degrees F. Readings were taken at five or six different rates, for each sand, at each water temperature, and the closeness with which the points plotted to a straight line was regarded as an indication of their accuracy. Typical curves from these readings for the eight uniform sands are shown as figure 3. These graphs have not been corrected to a uniform sand depth, and the different size loci do not represent the same water temperature. For any one sand curve, however, temperature is strictly a constant.

Five loss-of-head measurements calculated to 100 inches of depth and 125 m.g.a.d. rate for each of these sands, and the corresponding water temperatures, are shown in columns 2 and 3, respectively, of table 5. These figures are the result of about six hundred measurements each, of rate-of-flow, loss-of-head, and water temperature. These same data are shown graphically, as figure 4, the curves being naturally of the temperature-viscosity type. It was found that they could be represented by the equation:

$$l = \frac{a}{t + k}$$

where "l" = loss-of-head in feet and "t" = water temperature in degree F. In this expression "k" is theoretically a constant, although

when its value is calculated for each of these eight sands, variations occur, due to experimental error. The larger the number of sands employed, the more accurately can this value of "k" be determined; therefore ten other graded sands were selected and their head-losses measured at two different temperatures. (Refer to columns 2 and 3 of table 7.) By solving simultaneous equations for each of these ten sands, values of "k" were obtained, which, when added to those of the eight uniform sands, gave a mean value of 20.6. The above equation can then be written:

$$l = \frac{a}{t + 20.6} \tag{1}$$

EFFECT OF SAND SIZE

Using this equation the values of "a" can now be calculated for each of the eight uniform sands. Such values are shown in column 5 of table 5. They vary inversely with the corresponding sizes. By plotting on logarithmic paper the curve shown graphically as figure 5 is obtained, were the abscissas "a" represent sand size in millimeters, and the ordinates the calculated values of the variable "a".

The short horizontal lines drawn through each set of points (fig. 5) represent the complete range of possible size for each sand, that is, the limiting sizes according to the calibrations of the two sieves between which each sand is retained. The three different determinations of size (see table 1) for each sand are plotted on the corresponding line of limiting sieve sizes. It will be seen that the size values determined by the "count and weigh" method (shown by hollow circles on the graph, fig. 5) give the best straight line plotting of any of the three sets of values, with the single exception of the point for sand #1. When the locus was computed to fit these points, sand #1 was omitted from the calculation, because it is obviously affected by some other variable not influencing the other seven sands. This variable is that of porosity, which will be discussed later.

The equation of the curve, figure 5, gives the values of the variable "a" in terms of "s" (size), and can be written:

$$a = \frac{123}{s^{1.89}} \tag{2}$$

By combining equations (1) and (2)

$$l = \frac{123}{s^{1.89}\,(t + 20.6)} \tag{3}$$

This expression holds for the conditions under which it has been developed, that is, when the depth of the sand layer is 100 inches and the rate-of-flow is 125 m.g.a.d. Since the loss of head has been shown

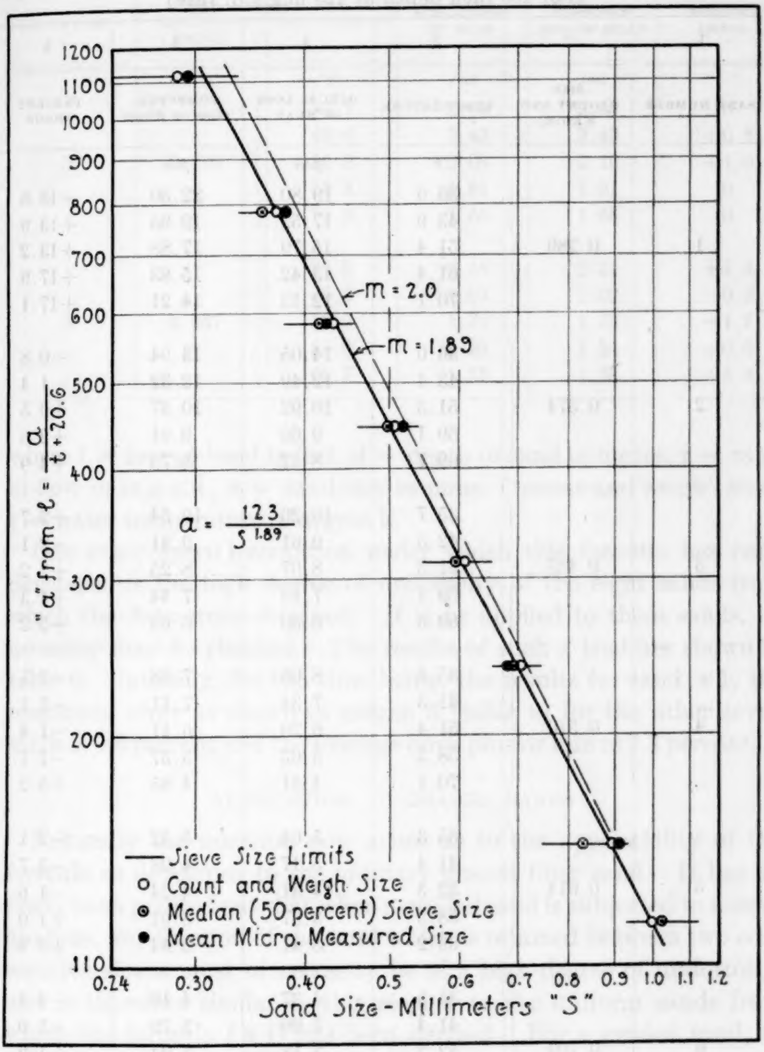

FIG. 5

to be directly proportional to both of these variables, they can be included in the equation, as:

$$l = \frac{9.84}{10^3}\left[\frac{d\ r}{s^{1.89}\ (t + 20.6)}\right] \qquad (4)$$

TABLE 6

Uniform Ottawa experimental sands—relation of computed loss of head (formula 4) to actual loss

(For 100 inch depth at 125 m.g.a.d. rate)

1	2	3	4	5	6
SAND NUMBER	SIZE (COUNT AND WEIGH)	TEMPERATURE	ACTUAL LOSS OF HEAD	COMPUTED LOSS OF HEAD	PERCENT ERROR
		°F.	feet	feet	
1	0.289	36.6	19.80	22.50	+13.6
		43.9	17.52	19.95	+13.9
		51.4	15.79	17.88	+13.2
		61.4	13.42	15.83	+17.9
		70.1	12.13	14.21	+17.1
2	0.374	36.0	14.05	13.94	−0.8
		43.4	12.49	12.32	−1.4
		51.3	10.92	10.97	+0.5
		59.1	9.66	9.91	+2.6
		69.7	8.32	8.73	+4.9
3	0.435	35.7	10.26	10.54	+2.7
		42.5	9.61	9.41	−2.1
		51.3	8.07	8.25	+2.2
		59.1	7.13	7.44	+4.3
		69.3	6.30	6.63	+5.2
4	0.510	35.5	8.06	7.83	−2.8
		41.3	7.34	7.11	−3.1
		51.4	6.20	6.11	−1.4
		58.3	5.63	5.57	−1.1
		70.1	4.61	4.85	+5.2
5	0.614	35.3	5.64	5.52	−2.1
		41.4	5.17	4.98	−3.7
		52.3	4.31	4.24	−1.6
		58.5	3.87	3.91	+1.0
		69.2	3.41	3.44	+0.9
6	0.709	35.5	4.37	4.19	−4.1
		41.4	3.99	3.79	−5.0
		52.2	3.18	3.24	+1.9
		58.8	2.88	2.96	+2.8
		69.4	2.60	2.60	0

TABLE 6—*Concluded*

1	2	3	4	5	6
SAND NUMBER	SIZE (COUNT AND WEIGH)	TEMPERATURE	ACTUAL LOSS OF HEAD	COMPUTED LOSS OF HEAD	PERCENT ERROR
		°F.	*feet*	*feet*	
7	0.899	35.1	2.69	2.69	0
		40.6	2.43	2.45	+0.8
		50.6	2.08	2.10	+1.0
		58.3	1.91	1.91	0
		70.2	1.66	1.66	0
8	0.997	34.8	2.18	2.21	+1.4
		40.6	2.03	2.02	−0.5
		50.2	1.73	1.76	+1.7
		58.9	1.53	1.54	+0.6
		69.7	1.31	1.38	+5.3

where l = loss-of-head in feet, d = depth of sand in inches, r = rate-of-flow in m.g.a.d., s = sand size in mms. ("count and weigh" size), t = water temperature in degree F.

One other special condition under which this formula has been developed is the high degree of uniformity of the eight sands from which the data were obtained. If it be applied to these sands, its accuracy may be checked. The results of such a trial are shown in table 6. Ignoring, for the time being, the results for sand #1, the maximum error as shown in colunn 6 (table 6) for the other seven sands is 5.3 percent and the average error plus or minus 2.3 percent.

APPLICATION TO GRADED SANDS

Naturally the question now arises as to the applicability of the formula as developed to the ordinary graded filter sand. It has already been pointed out that when a graded sand is subjected to a sieve analysis, any fraction of the sand which is retained between two consecutive sieves must of necessity be of a high degree of uniformity, and is therefore similar in this respect to the uniform sands from which the formula (#4) has been derived. For a graded sand, to determine the average size of the particles comprising each sieve fraction by the method of "count and weigh" would not only be tedious, but would also be subject to error with sand grains of very angular or irregular shape. For this reason it is necessary, and not illogical,

to accept the median 50 percent size between sieves as equally representative of the average size of the sand particles in the fraction retained between the two screens. One distinct advantage of adopting

TABLE 7

Relation of loss of head to porosity

For ten graded test sands at 125 m.g.a.d. rate and 100 inch depth

1	2	3	4	5	6
SAND NUMBER	TEMPERATURE	FEET LOSS OF HEAD		POROSITY COEFFICIENT	POROSITY (PERCENT VOID)
		Actual	Computed (formula 4)		
	°F.				
9	36.4	10.43	12.82	0.814	36.1
	60.6	7.58	8.99	0.843	
10	35.2	5.00	5.57	0.897	33.9
	60.5	3.34	3.83	0.871	
11	35.2	2.39	3.37	0.708	40.4
	60.5	1.74	2.32	0.750	
12	35.7	4.25	5.63	0.755	38.9
	60.6	2.94	3.90	0.754	
14	38.6	4.58	5.50	0.832	35.5
	60.8	3.27	4.00	0.817	
16	35.2	2.96	3.70	0.800	37.0
	60.7	1.99	2.54	0.784	
20	36.1	2.36	4.48	0.527	48.4
	60.5	1.62	3.13	0.517	
21	35.7	7.81	9.56	0.817	34.5
	61.3	5.76	6.57	0.878	
23	45.7	3.48	5.34	0.653	42.9
	60.6	2.91	4.36	0.667	
24	45.6	2.58	4.60	0.561	45.6
	61.6	2.12	3.70	0.573	

this parameter of size is that the median 50 percent sizes between sieves comprise a set of constants for any given calibrated series of screens.

Referring again to the sieve analyses of the sixteen graded sands as given in table 3, it may be assumed that these figures representing the percentages by weight found between screens, also represent percentages by depth in the case of a sand bed 100 inches deep. In other words, the sieve analysis percentages also represent depth in inches of each portion. On this basis, employing formula #4, it is quite possible to calculate the loss of head for each sieve portion, which is composed of very uniformly sized particles. The sum of these separately calculated losses, gives the total loss of head for the 100 inch sand depth. For any other given depth the loss is strictly proportional.

The results of such a computation are shown in table 7 for ten graded sands. Considerable discrepancy is apparent between the calculated losses (shown in column 4), and the measured losses (shown in column 3), the extent of which is indicated for each sand by a coefficient (column 5). This coefficient was obtained by dividing the actual measured loss by the calculated value. Multiplying the calculated loss-of-head value (formula #4) by this coefficient, the measured or actual loss is obtained. Representing this coefficient by "C_p", formula #4 may now be re-written to include it, as follows:

$$l = \frac{9.84C_p}{10^3}\left[\frac{d \cdot r}{s^{1.89}\,(t + 20.6)}\right] \tag{4a}$$

EFFECT OF POROSITY

Inasmuch as these coefficients show considerable variation, the actual head-losses must be further influenced by some variable so far not evaluated. This variable is the difference in shape exhibited by the grains, of which these different types of filter sands are composed. These differences are stated briefly in column #3 of table 2, but such a rough description does not make possible the necessary evaluation of the coefficient. It was necessary to find a reliable index of grain shape.

A serious attempt was at first made to find a suitable mathematical shape factor based upon microscopic examination of the sand grains. The attempt was not entirely without success, but finally the idea of employing such a method was abandoned, as certain to prove quite tedious and impractical. A measure of the effects of grain shape should serve the purpose as well, or better. It is quite apparent that

the porosity of a sand column must very greatly affect its resistance to water flow, and it is almost as obvious that porosity must largely depend upon grain shape. This simple index of shape was therefore adopted.

One of the immediate effects of a depature in grain shape from round to sub-angular or angular shape is an increase in its surface area. A sphere presents the minimum surface area of any geometrical solid, and therefore as the particle departs from the truly spherical shape its surface area increases. In the filter sand bed this should result in an increase in loss-of-head due to greater frictional area. Strangely, table 7 shows that the measured losses for all these filter sands are less than the calculated values, using the formula derived from round grain sand. Lesser resistance by the irregular shaped grains is indicated. This can only mean that the pores between the grains of angular sand are larger than those enclosed by round grains of the same measured diameter. If that be true, a measurement of the porosity of the graded sand column should indicate the value of the coefficient required.

Several methods of measuring porosity were tried in the attempt to find a method that would be practical, simple and accurate, and one that would not require special apparatus or laborious technique. The following method was developed, and is recommended as suitable.

The Jackson Turbidimeter outfit, found in nearly all water laboratories, provides a graduated, glass tube of approximately 2.8 cm. diameter by 75 cm. long. This tube is firmly clamped in a vertical position and half filled with water, to give a column of about 40 cm. height. A wide stem funnel is secured above the mouth of the tube, but not in contact with it. A representative, 400 gram sample of the sand is accurately weighed, and then poured in about ten successive fractions through the funnel into the tube, allowing each pour (of about 40 grams) to settle before the next is introduced. In the process of falling through the water column, each portion of sand will tend to grade itself hydraulically into ten miniature, stratified, sand layers. Throughout the pouring operation, extreme care should be exercised not to touch or jar the apparatus in any way to cause a false settlement of the sand column. A reading of the total volume of the sand is then made, and knowing its weight, the porosity can be calculated in terms of void percentage. It is suggested that the tube of about one inch internal diameter be used, as the value of porosity obtained varies slightly, using tubes of different diameters. Given the con-

stant 2.65 as the specific gravity of silica sand, the porosity may be calculated by the equation:

$$\text{porosity} = \text{per cent void} = 100 - \frac{37.7\,w}{v}$$

where "w" = weight of sand in grams and "v" = volume of sand in cc. as graded in tube.

As determined by this method, the porosity of the sand column in the tube is about 85 to 90 percent of the true porosity of the sand as it exists in the filter. The results by the tube determination, while relative, are fully applicable when the method is carefully followed.

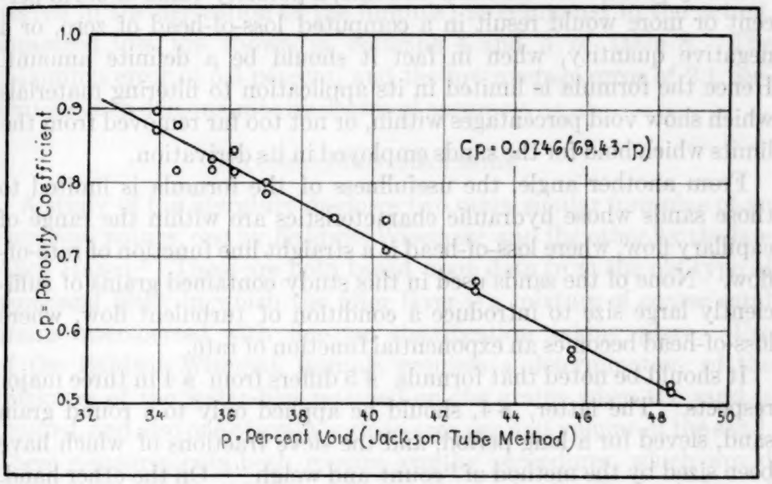

FIG. 6

Plotting the porosity, or void percent, as determined by the method given above, against the porosity coefficients, (refer to columns 5 and 6 of table 7) the straight line function shown graphically as figure 6 is obtained. The equation of this line gives the value of the coefficient in terms of porosity. Thus

$$C_p = 0.0246(69.43 - p)$$

where C_p = coefficient of porosity and "p" = porosity (%void) by Jackson Turbidimeter Tube method. This value of the coefficient "C_p" may now be substituted for it in equation #4a, giving:

$$l = \frac{24.2}{10^5}\left[\frac{dr(69.43 - p)}{s^{1.89}(t + 20.6)}\right] \qquad (5)$$

where "l" = loss-of-head in feet, "d" = depth of sand in inches, "r" = rate-of-flow in m.g.a.d., "p" = porosity (%void by Jackson Turbidimeter tube method), "s" = sand size in mms. (50 percent or median sieve size), and "t" = water temperature in degrees F.

APPLICATION OF FORMULA

An inspection of the porosity correction in this formula discloses the fact that a definite loss-of-head value would be calculated for a theoretical condition where the sand mass was impervious, and hence of zero porosity, whereas, in fact, the loss-of-head in such a case would be infinitely great. At the other extreme, a porosity value of 69.43 percent or more would result in a computed loss-of-head of zero, or a negative quantity, when in fact it should be a definite amount. Hence the formula is limited in its application to filtering materials which show void percentages within, or not too far removed from the limits which hold for the sands employed in its derivation.

From another angle, the usefullness of the formula is limited to those sands whose hydraulic characteristics are within the range of capillary flow, where loss-of-head is a straight line function of rate-of-flow. None of the sands used in this study contained grains of sufficiently large size to introduce a condition of turbulent flow, where loss-of-head becomes an exponential function of rate.

It should be noted that formula #5 differs from #4 in three major respects. The latter, #4, should be applied only to a round grain sand, sieved for a long period, and the sieve fractions of which have been sized by the method of "count and weigh." On the other hand, formula #5 is applicable to a sand of any grain shape, the sieve separation made with the usual 5-minute shaking and the size of each fraction assumed to be the median 50 percent size of the two limiting sieve calibrations.

In general, the application of formula #5 requires, then, only two laboratory measurements on any representative sample of graded filter sand, the hydraulics of which are in question. First, a sieve analysis carried out according to the standard procedure in common use, and a determination of porosity according to the method previously given. The results of the sieve analysis should be made to show each separated portion as percent by weight of the whole, these percentages then being applied directly to the depth, or thickness, of the sand bed to obtain the depth of each separate size-fraction. Having determined the porosity value, the loss-of-head is then calculated sep-

arately for each sieve portion, and the sum of these individual calculations taken as the loss-of-head through the entire sand bed. Inasmuch as the factors of rate-of-flow, water temperature, and porosity are constants in the calculation for all the fractions, the formula may be greatly simplified in practical use.

ACCURACY OF THE FORMULA

The formula has been applied to the sixteen graded filter sands of different kinds, types and gradings to show with what degree of accuracy these computations of loss-of-head may be made. The computed loss for these sixteen sands (six of which were in no way involved in the derivation of the formula) as compared to the actual, measured loss are shown in columns 4 and 5 of table 8. The maximum error of 5.5 percent, and the low average error of 2.1 percent serve to demonstrate its degree of accuracy.

CONCLUDING DISCUSSION

A study of the literature discloses two other similar formulae of an empirical nature, one by Baldwin-Wiseman[1] and the other by the late Allen Hazen.[2] These are both based upon data from the old type of slow-sand filter, in which the filter layer is a mixture of coarse sand grains interspersed with fine, no stratification taking place. Use of the Baldwin-Wiseman formula requires a determination of the thickness of the water-film retained on the grains when the sand is drained, and also one of grain-surface-area per unit volume of the sand. These measurements both require lengthy procedures, which would appear to be impossible to carry out on a hydraulically graded sand sample. Allen Hazen's formula specifies the use of the effective, or 10 percent size, and includes a coefficient "c", the value of which is dependent upon a number of variables, such as sand composition, grain shape, uniformity and compactness. This coefficient is said to range in value between 700 and 1000, but no method is given for determining its proper value under given conditions.

The writers transposed Hazen's formula and applied it to all of the 16 graded sands studied, with results as shown in columns 6, 7, 8 and 9 of table 8. The two extremes of Hazen's coefficient were used, since its value is indeterminate. The wide range of errors in these compu-

[1] "Statistical and Experimental Data of Filtration." Minutes of Proceedings Inst. of C. E. 1910.

[2] Report Massachusetts State Board of Health, 1892.

TABLE 8

Loss of head for sixteen graded test sands—actual loss compared to that computed by Hazen's formula and author's formula # 5

(For 100 inch depth and 125 m.g.a.d. rate)

1	2	3	4	5	6	7	8	9
		FEET LOSS OF HEAD						
			Computed—(authors' formula #5)		Computed by Hazen's formula			
					When C = 700		When C = 1000	
SAND NUMBER	TEMPERATURE	Actual loss	Feet loss	Percent error	Feet loss	Percent error	Feet loss	Percent error
	°F.							
9	36.4	10.43	10.36	−0.7	17.14	+64.3	12.01	+15.1
	60.6	7.58	7.27	−4.0	11.27	+48.7	7.90	+4.2
10	35.2	5.00	4.87	−2.6	7.70	+54.0	5.39	+7.8
	60.5	3.34	3.35	+0.4	4.94	+47.9	3.46	+3.6
11	35.2	2.39	2.41	+0.7	4.78	+100.0	3.35	+40.2
	60.5	1.74	1.66	−4.8	3.07	+86.4	2.15	+23.6
12	35.7	4.25	4.22	−0.8	7.58	+45.9	5.31	+24.9
	60.6	2.94	2.93	−0.5	4.91	+67.0	3.44	+17.0
13	38.7	5.74	5.86	+2.1	12.52	+118.1	8.77	+52.8
14	38.6	4.58	5.59	+0.2	9.29	+102.8	6.51	+42.1
	60.8	3.27	3.31	+1.2	6.37	+94.8	4.46	+36.4
15	38.6	4.96	4.95	−0.3	9.93	+100.2	6.96	+40.3
16	35.2	2.96	2.95	−0.3	5.23	+76.7	3.67	+24.0
	60.7	1.99	2.02	+1.6	3.34	+67.8	2.34	+17.6
17	35.3	3.32	3.41	+2.9	6.10	+83.7	4.27	+28.6
18	40.1	2.68	2.53	−5.5	4.63	+72.8	3.24	+21.7
19	35.8	3.50	3.65	+4.5	6.26	+78.9	4.39	+25.4
20	36.1	2.36	2.32	−2.0	6.23	+164.0	4.36	+84.7
	60.5	1.62	1.62	0	4.07	+251.2	2.85	+75.9
21	35.7	7.82	8.15	+4.2	15.75	+101.4	11.04	+41.2
	61.3	5.76	5.60	−2.8	10.10	+75.3	7.07	+22.7
23	45.7	3.48	3.49	+0.1	7.77	+123.2	5.44	+56.3
	60.6	2.91	2.85	−2.0	6.13	+110.6	4.29	+47.4
24	45.6	2.58	2.69	+4.4	6.55	+153.9	4.59	+77.9
	61.6	2.12	2.17	+2.4	5.09	+140.0	3.57	+68.4
25	45.6	3.60	3.69	+2.6	7.74	+115.0	5.42	+50.6
Mean error				±2.1		+87.8		+36.6
Maximum error				±5.5		+251.2		+84.7
Minimum error				0		+45.9		+3.6

tations, using Hazen's formula seem to show quite conclusively that the hydraulic characteristics of the slow-sand bed are quite different from those of the graded, stratified sand layer of a rapid filter.

It should be pointed out that the use of the formula which has been developed from this study is in nowise dependent upon guesswork. However, it is probably not entirely complete, or free from minor inaccuracies. Certain minor sources of possible error have been intentionally disregarded for the sake of simplicity and practical usefullness. One such example may be cited; a single, porosity value is determined and assumed to hold constant throughout the entire depth of the sand bed, which is obviously not strictly true, as reference to column 7 of table 1 will show. Note that here the porosities of sands numbered 2 to 8 do not vary more than 2.8 percent from the mean value of all seven; but that #1, the finest sand, varies about 7.5 percent from that mean. Incidentally this explains why measurements taken on uniform sand #1 were not included in the data used for the formulation of the size variable. Microscopic examination showed this fine sand to be composed of grains less round in shape than any of the other seven; which fact, it is believed, accounts for its greater degree of porosity. Assuming such to be the case, the assumption of a constant porosity for graded sands of irregular grain-shape must be a source of error; a minor source of error, however, because it would affect only one or two of the several sieve-fraction computations, and hence result in a very small percentage error of the sum-total.

In the beginning it was hoped that a single, definite size taken from a sieve analysis graph might be found, like Hazen's effective size, which would prove to govern the loss of head for the entire sand layer. This has been realized only in part. If the formula developed from these data be rearranged so that "s," the sand size, becomes the unknown, it will then be:

$$s = \sqrt[1.89]{\frac{24.2}{10^5}\left[\frac{dr(69.43 - p)}{l(t + 20.6)}\right]}$$

Applying this to a sand, the head loss through which has been measured, the governing size, "s," for that particular sand is found. In the case of a very uniform sand it closely approximates the 50 percent size, but as the uniformity of the sand decreases, this governing size is found to become smaller and smaller than the 50 percent size. This principle may be stated in another way by saying that the loss-of-head

is inversely proportional to the 1.89 power of the 50 percent size, in conjunction with a coefficient which varies as a function of the uni-

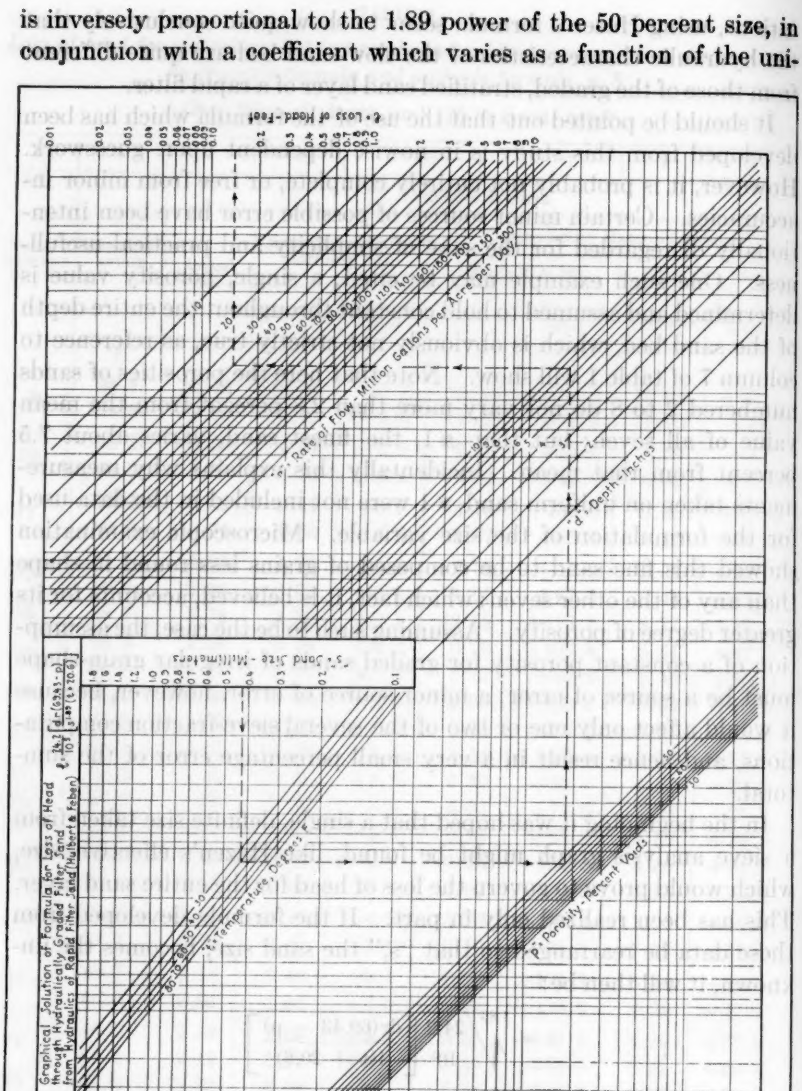

FIG. 7

formity. Again, this only holds true for a sand the sieve analysis of which gives a straight line graph on logarithmic-probability paper,

and any deviation from straight line plotting, especially at that end representing the fine sand, introduces considerable error. Unfortunately, filter sands which give a straight-line grading curve are rare, and such a formula would then have serious limitations.

A further worthwhile investigation may be suggested. Slichter[3] gives a theoretical basis for assuming that the loss of head through sand is inversely proportional to the square of the sand size, assuming that the grains are all perfect spheres. Microscopic examination of the eight uniform sands used in this study, reveals that the larger sizes are those which most closely approach a truly spherical shape. Referring again to figure 5, it will be noted that a dashed-line curve has been drawn which intersects the point representing sand #8, the largest of these sands, and the most nearly spherical grain. This curve has been given a slope value of 2, and it is to be noted that all the points representing the count and weigh sizes are displaced from this curve by a series of values which appear to be definitely related to the porosity measurements of these sands (table 1, column 7). This indicates the possibility of developing an expression containing a variable exponent of the power of "s," one which would vary inversely with the porosity. It was found that this study did not provide sufficient data to warrant a formulation on this basis.

The writers wish to express their appreciation of the encouragement and coöperation afforded throughout this work by George H. Fenkell, General Manager, Arthur B. Morrill, formerly Assistant Engineer of Filtration, and Wm. M. Wallace, Superintendent of Filtration, Detroit, Mich.

(Presented before the Central States Section Meeting, September 23, 1932.)

DISCUSSION

H. E. Babbitt (University of Illinois, Urbana, Ill.): The work done by the authors in the development of their formula for the loss of head through a rapid sand filter has resulted in an expression which is of practical value. The thorough and scientific methods which were used in its derivation make the result convincing and do great credit to the authors. It is unfortunate that the empirical nature of the formula confines its usefulness within the rather narrow limits under which it was derived. This, however, is an inherent difficulty with

[3] Motions of Underground Waters. U. S. Geological Survey, Water Supply Paper No. 67, 1902.

all empirical formulas. The results to be obtained by the application of this formula, within its proper limits, are probably more accurate than can be obtained by the use of any other formula so far developed.

If space had been available it might have been desirable for the authors to show more clearly the method of applying the formula to the solution of practical problems. Possibly some illustrative examples would have been helpful in this connection. Practice today specifies filter sand in terms of the "effective size" and many reports on the operation of filters include a reference to this term. The term is, therefore, still of value in spite of the fact, brought out by the authors and other investigators, that the ten percent size is not properly the effective hydraulic size. However, since experience has been based on sand expressed in this term the relation between the results found by this formula and the old term "effective size" would greatly enhance the value of the results to be obtained by the use of the formula. The authors have appreciated this fact as is demonstrated by their unsuccessful attempt to find some "size" which might properly express the hydraulic effectiveness of any particular sand. The impossibility of doing this has been clearly demonstrated.

In view of the radical change in practice which would be required by the use of this formula in the specification of filter sand or in the correction of the construction of existing filters it is probable that its adoption will be slow until the results obtained from it can be expressed in terms of practical experience. This is a problem which obviously could not be solved by the authors. They have presented the profession with a formula which tells the designer how the loss of head through his filter will be affected by the size of sand he uses. It will require some years for practice to determine the desirable loss of head and the desirable proportions of coarse and fine grains for the best results in operation. These desirable conditions can be more quickly studied and expressed now that so acceptable a formula has been presented.

Charles R. Cox (State Board of Health, Albany, N. Y.): The authors of this valuable and timely paper are to be congratulated upon the logical and clear discussion of their comprehensive study of the hydraulics of filter sand. This investigation is unusual in being made by municipal employees rather than by the staff of a technical school. The City of Detroit is to be congratulated for its vision in encouraging these basic studies.

The reader of this paper will realize at once that carefully controlled and extensive experimental work is under discussion. It is significant that consideration has been given to the several sand sizes making up any one filter sand, rather than overemphasis being given to two statistical measures of sand size, namely, the "effective size," and "uniformity coefficient." It is evident, for instance, that two distinct filter sands may have the same "effective size" and yet varying characteristics due to different "uniformity coefficients," or vice versa. This factor is realized by careful investigators in this field, who study the complete data secured by the sieving of sand, and do not place undue weight upon the single measure, "effective size." Their final conclusion, however, is a matter of judgement, because while the "effective size" may be a practical measure of the suitability of sand for use in filters, nevertheless, it is impossible to correlate this single value with filter behavior, such as loss of head or washing characteristics. The study under discussion, therefore, is greatly needed to provide the basic data which correlate the actual size of *all* portions of the sand bed, porosity of the sand, depth of sand bed, rate of filtration, water temperatures, etc.

The basic formula resulting from this study may be used in a practical way in studying the behavior of filter sand, where due weight is given the various factors involved, notably the various sand sizes making up a sand bed. This latter factor has been overlooked by many workers who fail to realize that a rapid sand filter bed is not a heterogeneous mixture of sand of different sizes, but a hydraulically graded bed with different layers of sand, each of fairly uniform size. Experience has shown that the upper fine layer is of very great importance in filter bed behavior, and hence knowledge of the characteristics of this layer, gained by the procedure under discussion, is of great assistance. In fact the partial success of the "effective size" measurement is due to the fact that this size roughly represents the finer portion of the sand mass, which experience has shown is the "effective" portion of the bed.

The formula under discussion should be tested with filter sand which has been utilized in a filter, because the summation of the losses of head of two mixtures of clean, unused sand may be the same, while quite different values would be secured from the same two sands after use, because of variation in adsorption of fine material, and in the effectiveness of washing of different sands under the same hydraulic conditions. The basic formula deduced, therefore, applies primarily

to clean sand and to the initial loss of head through the sand. This does not detract, however, from its usefulness, because it permits a study of the influence of adsorptive material in modifying the loss of head of various filtered sands, which are first tested when clean, as outlined in the paper, and then after use.

One of the most significant factors discussed in the paper is the porosity of sand beds. Too much weight seems to have been given to sand size in the past, and not to the actual hydraulic characteristics of the sand, which depend as much upon porosity as upon grain size. The porosity obtained by the laboratory procedure outlined in the paper, however, must be used with care, because the porosity of a sand as tested may be quite different from that of the same sand in a full scale filter bed after being washed in the conventional manner. This again does not detract from the basic formula, but indicates the usefulness of the formula in studying sand bed behavior.

It is hoped that this paper will be widely read and that the sand sizes and porosity of numerous filter sands will be determined by the technique indicated in order that the total loss of head may be calculated and compared with the actual loss of head in full scale filter beds. Such comparisons will disclose many interesting facts as to the hydraulics of filter sands, gravel beds, and underdrains. In this way the laborious work of the authors will be rewarded, and their contribution to water works practice will bear fruit in improved filter performance.

Wm. M. Wallace (Superintendent of Filtration, Detroit, Mich.) The writers of this paper are to be highly commended for their work as they have made a distinct contribution to the art of water purification. The formula developed in this paper is not only an aid to the designer, but should also be of considerable assistance to the filtration plant operator. It is also of interest as a necessary preliminary to studies of the rate of increase of loss of head in ordinary operation.

The first inclination of an interested operator will be to see whether the initial losses actually measured in his filters agree with those calculated from the formula. Several precautions are necessary in making this comparison. Most operators have a screen analysis of their sand, but frequently these have not been made with calibrated sieves. Considerable errors are apt to result if sieves are not calibrated. The analysis should also have been recently made, as the sand characteristics may have changed through washing out of fine sand or building up of the grains.

It is to be remembered that the loss shown by the formula is that through the sand only, whereas the value ordinarily obtained in an operating plant includes the loss in the strainer system and in that part of the effluent piping up to the entrance to the rate controller. By inserting a piezometer tube into the coarse gravel layer it may be possible to measure directly the loss in the strainer system and the effluent piping.

For any given filter it would be possible to prepare a table showing what the total initial loss of head should be for any rate of filtration and water temperature. If the routine observations of initial loss show variations from the values indicated by the table, the cause of the discrepancy should be sought. This might be due to air-binding, to the fact that the filters were not being washed clean or to some other difficulty that required attention. Thus changes in the initial loss of head could be made to serve as warnings of unfavorable conditions that may develop in the filters.

If the operator finds that the initial loss in his filters is about what would be expected from the formula, but if this loss is so great as to be objectionable, the formula gives him a way of estimating just what needs to be done to the filters to reduce the initial loss to any desired extent. A calculation will show what thickness of fine sand must be scraped from the top of the filter to bring about this result. Whether such scraping would have an unfavorable effect on the efficiency of filtration is a matter that must be determined by experiment or the good judgment of the operator.

The evaluation of the effect of temperature is interesting. It will be a surprise to many to learn that the initial loss of head in the sand may be increased more than 50 percent from summer to winter from this cause alone. Calculation will show in such cases that changes which might otherwise indicate trouble in the filters are simply due to changes in the viscosity of the water.

Arthur B. Morrill (Formerly Assistant Engineer of Filtration, Detroit, Mich.): In this paper the authors have taken an important step toward the solution of a problem of basic interest to the filter designer. It is surprising that no previous attempt has been made to establish such fundamental data. It is to be hoped that the proposed attempt to study the more complicated hydraulics of a clogging sand bed will be as successful as has been this work with clean sand and pure water. If it is, it will go a long way toward changing the specification of filter sand from a matter of guesswork to a rational process.

The very great amount of painstaking research which lies back of this paper is not apparent without careful reading. To reduce the paper to convenient size much of the supporting discussion and the details of calculation have been omitted. This necessarily makes the study a little more difficult to follow. It will be helpful for the reader

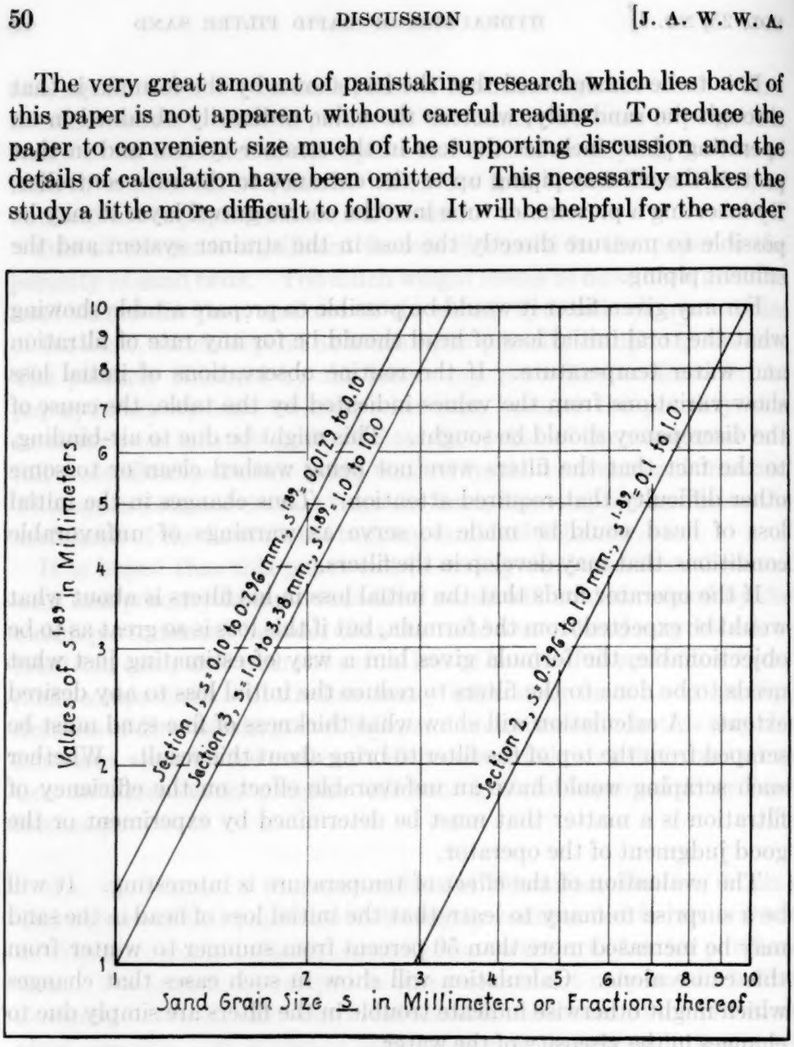

Fig. 8. Diagram for Raising Sand Grain Sizes to 1.89 Power

to carry out for himself the calculation necessary to obtain one of the values given in column 4 of table 7.

The formulas in this paper look a little formidable and the ease with which they may be applied is not at first apparent. The only thing which is at all troublesome is the raising of the sand grain size s to the 1.89 power. This can be done quickly on a log log slide rule, but is even simpler with the help of figure 8.

The calculation for sand No. 14 is particularly interesting, as this is one of the least uniform of all the sands. Calculating for the temperature 38.6°F. and a rate of 125 m.g.a.d., formula (4) reduces to:

$$l = \frac{9.84}{1000} \times \frac{125\ d}{59.2\ s^{1.89}} = \frac{.0208\ d}{s^{1.89}}$$

The details of calculating the total loss of head through a bed of graded sand of 100-inch depth are shown in the following table:

PORTION NUMBER (SEE TABLE 3)	MEDIAN SIZE "s"	$s^{1.89}$	DEPTH "d"	INITIAL LOSS OF HEAD
	mm.		*inches*	*feet*
2	1.59	2.403	5.90	0.051
3	1.23	1.479	5.00	0.070
4	1.02	1.038	14.6	0.293
5	0.83	0.703	18.2	0.538
6	0.70	0.509	13.1	0.535
7	0.60	0.380	12.5	0.685
8	0.50	0.270	15.8	1.217
9	0.42	0.194	7.70	0.826
10	0.36	0.145	4.00	0.574
11	0.28	0.090	3.16	0.730
12	0.15	0.028	0.04	0.030
Total........			100.00	5.55

The value above obtained for the total initial loss of head, 5.55 feet, is to be compared with the value 5.50 feet, shown by the authors in table 7. The points of interest in connection with this calculation are these: Considering the three finest portions of the sand, they amount to 7.20 percent of the total depth, but are responsible for 24.1 percent of the total loss of head. At the bottom of the bed, on the other hand, the two coarsest portions amount to 10.9 percent of the total depth but are responsible for only 2.18 percent of the loss of head.

These relations are due to the fact that the loss of head varies inversely nearly as the *square* of the sand size. Of two portions of equal thickness, if the grain diameter of one portion is half that of the other, then the first has a loss of head 3.71 times as great as the second. Thus the fine portions of the sand have a disproportionately large effect in causing resistance to flow and the coarsest portions have comparatively little effect.

It seems probable that under ordinary conditions the sizing of the

coarser 90 percent of the sand of a rapid sand filter has little or nothing to do with its efficiency in removing turbidity and bacteria. Even of the finer 10 percent, the fineness of the top tenth may be as important as that of all the other nine percent combined. Thus what may be called the "top size," that is the average size of the finest one percent of the sand, probably comes nearer to determining the efficiency of filtration in rapid filters than the effective size as defined by Hazen.

It can be seen that if the top size of the sand in a filter is finer than necessary it will have an important unfavorable effect in increasing the initial loss of head. The effect of this unnecesarily fine sand in increasing the clogging rate, that is in decreasing filter runs, has not yet been studied quantitatively. If it follows any law similar to that for initial loss of head, then the presence of this fine sand is doubly objectionable.

As has been suggested elsewhere,[4] the lower two-thirds of the sand in an ordinary rapid sand filter probably serves no filtration purpose, merely acting as the physical means of supporting the effective upper third. Application of the formula developed in this paper lends support to this view. The relatively slight hydraulic resistance of the lower part of the sand bed makes it seem improbable that it has any important effect on filtration. This suggests that in the interest of economy the supporting layer should be made as thin as the physical requirements will permit.

M. G. Mansfield (Division Engineer, Morris Knowles Incorporated, Pittsburgh, Pa.): The authors have made a careful and exhaustive investigation of the hydraulics of filter sands which to date has been more or less theoretical. Their next investigation, however, will probably deal more with the practical side of water purification or with chemically coagulated water, and will be of special interest as it will show how the theoretical results can be applied in practice.

They have devised a new formula for the loss-of-head in filters which, as formulae have in the past, considers the viscosity of water, the voids or porosity of the sand, and the sand size. Dr. Charles Terzaghi, formerly of Massachusetts Institute of Technology, has devised a formula which I believe has considerable merit and will

[4] "Research Aids Economy in Filter Plant Design," Arthur B. Morrill, Engineering News-Record, April 28, 1932, p. 623.

check the experimental results closer than the older formulae of Baldwin-Wiseman and Allen Hazen. Dr. Terzaghi has described this formula in Engineering-News Record, Volume 95, No. 21, page 832. His formula has the objection that it contains a constant which has to be determined or estimated before the formula can be used. The greatest factor influencing this constant is the quality of the grains and, as most filter sands are polished and rounded, therefore, there should be little variation in this constant, when dealing with such sands.

The authors state that they found considerable variation in the value of "k" in their formula for the influence of temperature on the loss-of-head in filter sands. It has been pretty well determined, experimentally, that the loss-of-head in sand varies directly as the absolute viscosity. The authors' formula can be used to check the viscosity of water curve approximately, but the error, even with ordinary ranges of water temperatures, is as high as 6 or 7 percent. As a substitution for the authors' formula of

$$l = \frac{a}{t + 20.6}$$

it is suggested that the following formula be tried to see if some of the variations can be eliminated:

$$l = 0.00242\, a \frac{500 - t}{25 + t}$$

This formula will check the temperature-viscosity curve of water for ordinary ranges, with an error of less than one percent. The suggested formula was derived from a temperature-viscosity of water table by the methods given in Lipka's book, "Graphical and Mechanical Computation."

I would like to make one suggestion to the authors for their consideration in future work. In the operation of a filter, by far the most important portion of the sand as regards purification is the upper six inches. Practically all of the loss-of-head in a dirty filter is encountered in this portion of the bed and likewise most of the purification during the entire operation of the filter occurs here. It would appear wise, therefore, to concentrate investigations on this portion of the bed. The information needed by filter plant designers is the influence of the characteristics of the sand in the upper 6 inches on the amount of purification effected and the length of run. The size of

this portion should be a compromise. It should be small enough so that the purification is adequate and large enough so that the runs are not unduly short.

J. F. Laboon (Of John N. Chester, Engineers, Pittsburgh, Pa.): To the practising engineer who usually has but limited opportunity for research work it is most gratifying to hear such a paper as has been presented by Messrs. Hulbert and Feben. Deserving credit should be offered them, not forgetting at the same time the benevolence of the heads of the Detroit Water Works Department who have made possible through earnest support and encouragement the experiments covered by the paper and to whom, therefore, the engineering profession also is indebted in a great measure.

The paper by its findings will serve to give the operating chemist or engineer a better understanding of the reasons for fluctuating initial losses of head in rapid sand filters with clean filters, but to the designing engineer other factors enter into the selection of the proper size of filter sand besides loss of head due to the size, depth and character of filter sand under varying temperatures.

The prime purpose of a filter is to produce a satisfactory effluent from the standpoint of bacteria and clarity or turbidity and these elements are dependent upon the design of other units of the treatment plant as well as the filters and also upon efficient chemical treatment.

Since reading the paper I have been trying to apply the data given therein in a practical way to the economics of filter design but I am frank to admit I have been unable to do so upon first reading and initial study for the time being, except to verify by fact what engineers have long believed through accepted practice because it worked. Further study may develop such features as may be of considerable value in design but at least it may be said that the authors have made a valuable contribution to the studies already made by Hazen and others. It is hoped these studies will be continued to other features of rapid sand filter hydraulics until the whole story is finally completed.

The data given in the paper make apparent readily that it is not advantageous to specify and use a sand of absolutely one size such that the uniformity coefficient approximates unity, because the initial loss of head for any given grading is greatest as the uniformity coefficient approaches unity. This is valuable knowledge in the selection of filter sands.

Likewise, it would appear that an angular grained sand is more de-

sirable than a round grained sand of corresponding grading for the same reason. This, too, is of value to the engineer.

How far we should go in the direction of greater uniformity coefficient values and thus reduced initial loss of head, involves other factors of filter plant operation, which it is hoped may be studied in the laboratory to a definite conclusion. It is possible that the data given in the author's paper in conjunction with other data which may be revealed in an extended study of filter hydraulics may make possible definite economies in filter design through reduction of depths of gravel and sand beds, depths of filter tubs and a consequent reduction in operating costs by reduced loss of head as affecting the levels of clear water storage reservoirs and pumping heads where high service pumps are required.

Upsetting sand and gravel beds in filters and wash water distribution throughout the filter are little discussed, but are uppermost in the mind of the operator and designing engineer and therefore deserving of attention and full study. Little is said of upsetting sand and gravel beds, but many filters suffer from this failing. It is noted that the authors have tied down the gravel by means of a screen presumably to facilitate changing of the sand bed but in design we have returned again to the practice of tying down and anchoring the gravel bed which practice was discontinued fifteen years ago. This is to prevent the upsetting and disintegration of the graded layers of gravel and the serious disturbance of the overlying sand.

H. G. Turner and G. S. Scott (Director and Assistant Director of Research, Anthracite Institute, State College, Pa.): So far as we have been able to determine, Hulbert and Feben's paper is the first of its kind, and fills a definite need in engineering data. The older data were so approximate, because of insufficient consideration of the points brought out by Hulbert and Feben, as to be little more than a guess in some cases. This is well illustrated in the final tabulation by Hulbert and Feben, where initial losses of head, as calculated by the Hazen formula are shown compared with the actual and very accurately measured losses. The "factor" of the Hazen formula is a compound factor made up of a number of component factors. Inasmuch as the factor in the Hazen formula must be assumed, there is no reason to believe that the minimum errors shown in the tabulated comparison would be representative of actual errors made by the use of the Hazen formula.

We have been doing some experimental work on Pennsylvania anthracite at State College, School of Mineral Industries, along the lines followed by Hulbert and Feben, except that we did not go into the matter as thoroughly from an experimental point of view. The work at State College was undertaken in order to answer frequent questions concerning the initial loss of head through beds of Pennsylvania anthracite, which is being considered very favorably as a filter medium for reasons that do not properly belong in the present discussion. It seemed desirable to express the results in terms already in common use, and as Hulbert and Feben's work was unknown to us at the time, we were limited to the Hazen formula, which, of course, is too indefinite. It was recognized at the start that the flow of water through a bed of broken solids depended upon the nature of the entire sizing curve, as clearly brought out by Hulbert and Feben's summation of losses of head, and not upon a single point, as effective size, except where the sizing curves might all be exactly alike, and where the shape factor would remain the same. Within certain limits, the uniformity coefficient may be considered equivalent to a second point on the sizing curve, so that, for the same material, and within a range in effective size from 9.5 to 0.44 mm. and uniformity coefficient up to 1.6, the loss of head may be approximately expressed as a function of these two terms.

Our experimental results verify the very high degree of accuracy of the work of Hulbert and Feben. The approximate formula, derived to fit the experimental data obtained at State College, when applied to the experimental results obtained by Hulbert and Feben, show approximately equal positive and negative errors over the range of overlap, which is at least, an excellent confirmation of the experimental work.

Figure 9 shows the results of Hulbert and Feben plotted as circles and our results plotted as squares on logarithmic paper after the manner of Chilton and Colburn, (Industrial and Engineering Chemistry, 23, page 914, August, 1931). The straight line represents an average of experimental results for flow of fluids through pipes packed with broken solids of various kinds and sizes, lead shot, etc. The ordinate, f′, is the modified friction factor, calculated from the diameter of particle, the pressure drop (or head loss), the length of column (or depth of bed), the velocity of the fluid flowing, and the acceleration due to gravity. The abscissa is the modified Reynolds number, where d is the diameter of particle, u is the velocity of fluid flowing, rho is the

density of the fluid, and mu is the viscosity, all in English units. It will be noted that all points fall below the line, indicating that the void spaces between the particles are greater after classification by backwashing than in the case of closely packed solids represented by the straight line. The solid circles are the eight experimental sands, which, due to uniformity, fall in a straight line. The graded sands, due to differences in porosity, sizing analysis, etc., are more spotted. This spotting does not, as will be noted, indicate experimental error,

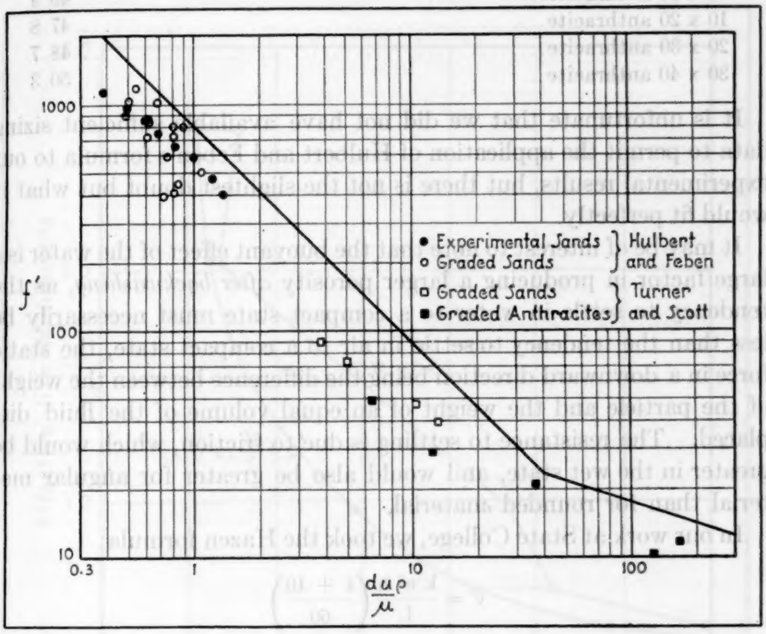

FIG. 9

but does show that the data from which the points are calculated does not include all the factors entering into the case. The open squares are for graded sands tested at State College, and the solid squares are for Pennsylvania anthracite. Two of the points for anthracite appear in the turbulent range of flow (to the right of the break in the line, viscous, or straight line flow, obtaining to the left of the break).

It will be noted that the points representing our results are a little more to the left of the line than those of Hulbert and Feben. This

indicates a greater porosity for the materials tested at State College. A recent series of tests to check up on this point, made in a 100 cc. cylinder, approximately one inch in internal diameter, gave the following results:

Mesh	*Percent porosity*
20 x 30 Ottawa sand	44.3
20 x 30 graded sand	44.5
30 x 40 graded sand	45.0
4 x 8 anthracite	47.1
8 x 10 anthracite	49.4
10 x 20 anthracite	47.8
20 x 30 anthracite	48.7
30 x 40 anthracite	50.3

It is unfortunate that we did not have available sufficient sizing data to permit the application of Hulbert and Feben's formula to our experimental results, but there is not the slightest doubt but what it would fit perfectly.

It may be of interest to note that the buoyant effect of the water is a large factor in producing a larger porosity *after backwashing*, as the tendency to settle in water to a compact state must necessarily be less than the tendency to settle in air to a compact state, the static force in a downward direction being the difference between the weight of the particle and the weight of an equal volume of the fluid displaced. The resistance to settling is due to friction, which would be greater in the wet state, and would also be greater for angular material than for rounded material.

In our work at State College, we took the Hazen formula;

$$v = \frac{k\ e^2\ h}{L}\left(\frac{t + 10}{60}\right)$$

put v equal to the rate of flow in gallons per square foot per hour, e equal to the effective size in millimeters, h equal to the loss of head in inches of water, L equal to the depth of bed in inches, and t equal to the temperature in degrees Fahrenheit, and calculated the numerical values of k under the various conditions. The values of k were then plotted against effective size, as shown in figure 10, by the irregular line marked "e vs k." The values of k so obtained were then divided by the uniformity coefficients and the quotients plotted against effective sizes. The smooth hyperbola, marked "e vs $\frac{k}{c}$," where c is the uniformity coefficient, was then obtained. The equa-

tion of this hyperbola was then determined, solved for k, and the value of k substituted in the original equation given above. This resulted in a general equation of the form

$$\text{Rate} = (\text{constant}) \frac{c\ e\ h}{L} \left(\frac{t + 10}{60}\right)$$

with notations as above, and with the constant determined largely by the shape of grain.

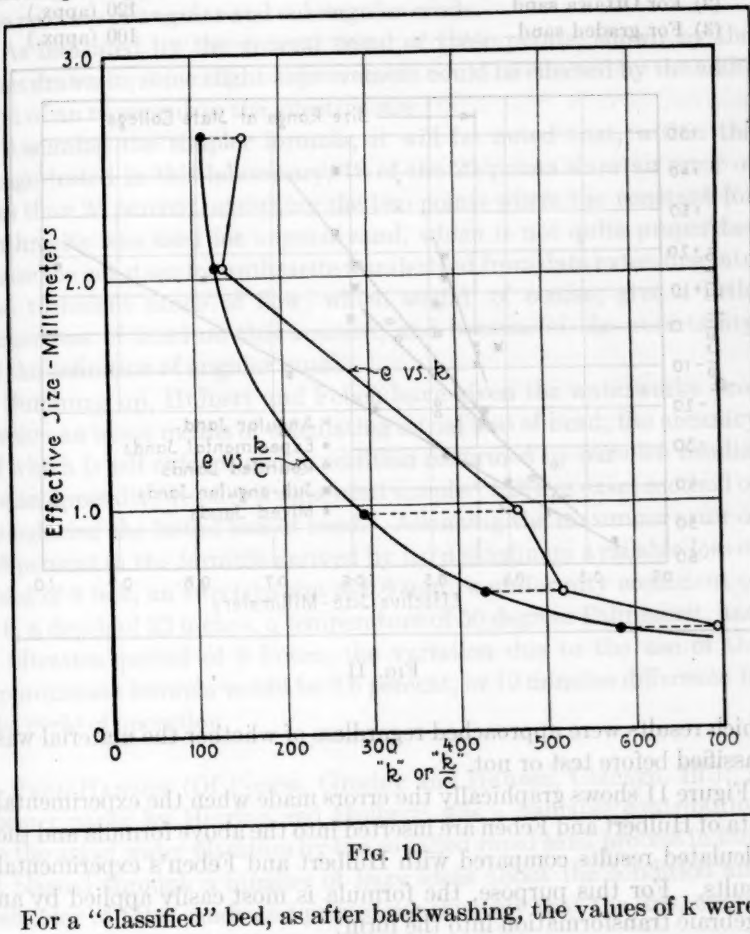

FIG. 10

For a "classified" bed, as after backwashing, the values of k were:

(1) For Pennsylvania anthracite 520
(2) For Ottawa sand 470
(3) For graded sand 440

For a closely packed bed, as when the sand or coal is put in dry, as in a slow sand filter:

(1) For Pennsylvania anthracite 263
(2) For Ottawa sand 280
(3) For graded sand 236

For the worst air bound condition we were able to get:

(1) For Pennsylvania anthracite .... 120 (appx.)
(2) For Ottawa sand .... 120 (appx.)
(3) For graded sand .... 100 (appx.)

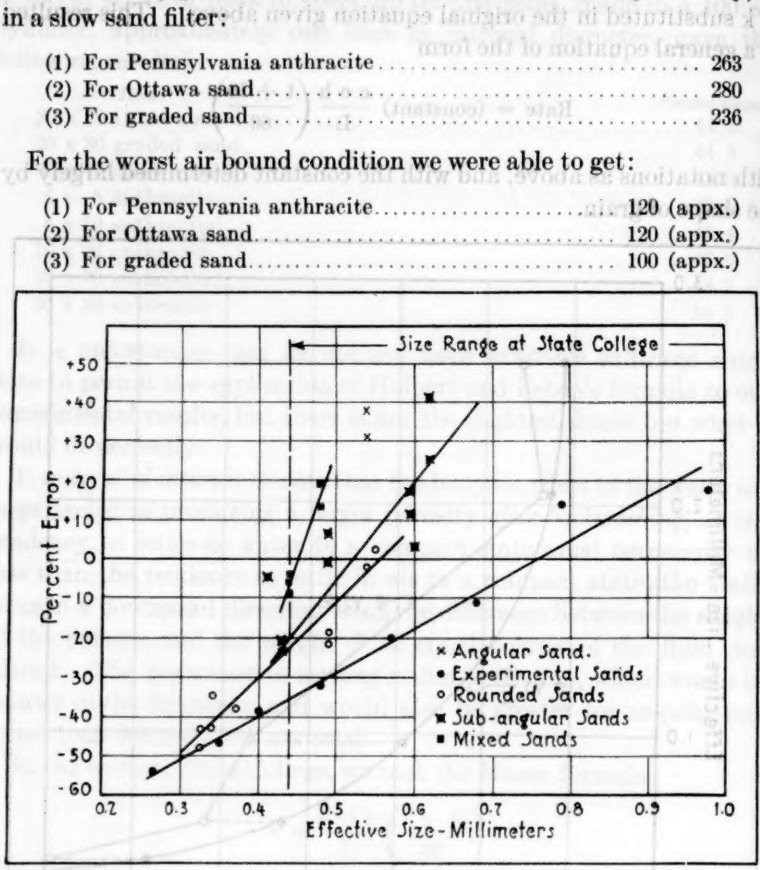

FIG. 11

which results were approached regardless of whether the material was classified before test or not.

Figure 11 shows graphically the errors made when the experimental data of Hulbert and Feben are inserted into the above formula and the calculated results compared with Hulbert and Feben's experimental results. For this purpose, the formula is most easily applied by an algebraic transformation into the form;

$$h = \frac{\text{Rate} \times \text{Depth}}{12\ c\ e\ k}\left(\frac{60}{t + 10}\right)$$

where h is now the loss of head in feet. The standard rate of 125 mgad equals 120 gallons per square foot per hour, the latter being substituted in this formula. Depth is in inches, and other notations are as above. The constant for Ottawa sand (well rounded), of 470 was taken for the "rounded" sands. The constant, 440, was taken for the subangular sands, as our "graded" sand approximated this condition. The constant, 520, (for anthracite) was taken for the angular sand (Pennsylvania anthracite being angular), and 480 was assumed for the mixture of angular and subangular sands.

As indicated by the general trend of these points, shown by the lines drawn in, some slight improvement could be effected by the addition of an exponent to the effective size.

Assuming the simpler formula, it will be noted that, within the range tested in this laboratory, 18 of the 20 points show an error of less than 25 percent, excepting the two points where the constant for anthracite was used for angular sand, which is not quite proper because the constant for anthracite was derived from data extending into the turbulent range of flow, which would, of course, give a little larger loss of head on this account, and because of the uncertainty of the definition of angular sand.

Summing up, Hulbert and Feben have given the waterworks profession an exact means of calculating initial loss of head, the accuracy of which is self evident and in addition confirmed by our own results. As an appendix, we offer a somewhat simpler, but less exact method of calculating the initial loss of head. Assuming the maximum error of 25 percent in the formula derived by us, a maximum available loss of head of 8 feet, an effective size of 0.6 mm., a uniformity coefficient of 1.6, a depth of 32 inches, a temperature of 50 degrees Fahrenheit, and a filtration period of 8 hours, the variation due to the use of the approximate formula would be 2.6 percent, or 12 minutes difference in the cycle of operation.

Paul Hansen (Of Pearse, Greeley and Hansen, Chicago, Ill.): A former paper by Herring and Hulbert[5] was a valuable contribution to the art of water filtration by the so called rapid sand process in that it blasted through a number of hazy ideas about the condition and behavior of filter sands and proved conclusively that the dirt, coagu-

[5] Studies of the Washing of Rapid Filters, Roberts Hulbert and Frank W. Herring. Journal of the A. W. W. A., November, 1929, pages 1445–1506.

lant and biological substances adhering to filter sand constitute a hindrance and a disturbing element in filtration and not a help or partial help as had hitherto been supposed. Of course, objectionable effects of excessive coating of sand grains had been clearly recognized but it was general belief that the sand grains must not be thoroughly clean.

These authors went further and established the possibility of keeping a sand bed clean by a proper adjustment between wash water rate and size of sand grain.

These experiences raised questions in the minds of Hulbert and Feben as to whether there is a clear understanding of the fundamental behavior of filter beds in the rapid sand filtration process and they have set about throwing light on these questions by studying the hydraulic properties of filter beds of clean sand, passing clean water. They have adopted a technique that is skillful and direct and have not made the mistake of trying to find out too many things at one time.

The points that the authors have established may not have any immediate effect on improving filtration practice, but they have laid down a valuable ground work both in technique and facts determined. Some of the items of interest brought out in the paper may be summarized and commented on as follows:

They have shown a straight line relation between depth of sand and loss of head and between rate of filtration and loss of head.

They have established an accurate relation between loss of head and temperature. Such relations were approximated in a number of filter plants as a result of the observations of the operators, but this paper presents the first carefully conducted experimental work on sands as used in rapid sand filters.

The most remarkable accomplishment of the authors is the development of a formula whereby the loss of head of a sand bed as arranged in a rapid sand filter may be calculated with surprising accuracy from the sieve analysis and a relatively simple determination of the porosity of the sand. The accuracy of this method contrasts strongly with the inaccuracy of Hazen's method which was developed for use with unstratified sands as used in slow sand filter beds. The method of Hulbert and Feben bids fair for general adoption in the examination of filter sands. The necessary computations are greatly facilitated by a convenient diagram prepared by the authors.

The studies indicate that the use of the median size of sand grain is more helpful in considering hydraulic properties of filter sand than

effective size or uniformity coefficient. Whether this will hold true in considering the filtering behavior of sand remains to be seen. Light may be thrown on this question by applying filtered water containing known quantities of coagulant to filter sands of known hydraulic and physical characteristics.

From the hydraulic studies made by the authors one cannot draw conclusions as to the type of sand best suited to rapid sand filters, but with improved measuring devices for hydraulic qualities the foundation is laid for finding answers to such questions as:

"How are the hydraulic qualities of a filter sand modified in actual filtration?"

"Is a uniform sand important to filtration, and if so what degree of uniformity is desirable?"

"Is a round or angular grain preferable in rapid sand filtration?"

"What size grain is best adapted to good filtration and adequate cleansing of the sand bed?"

Gale Dixon (Youngstown, Ohio): I have read this very interesting paper by the two gentlemen from Detroit, but I have not given it sufficient study to warrant going into a discussion of it at this time. I do, however, wish to congratulate these young men and thank them for presenting this very able and thorough study of a somewhat difficult subject. It is exactly this sort of painstaking investigation that we need in a very great many details in the civil engineering profession.

The paper has all the ear-marks of the same sort of quality which distinguished the paper presented by Hulbert and Herring three years ago. That was a little closer home to most of us because it attacked a subject concerning which all of us had had visual experience in the practical operation of filters.

I hope that this document will prove just as valuable to the profession as the earlier one.

There is one question I would like to ask. I have never done any experimentation of exactly this sort, but have made some hydraulic experiments, and sometimes you find some funny things. I would like to ask if it has been demonstrated that the size of the tube in which this experiment has been made has no confusing effect upon the results.

As I understand it, this experiment was done with a tube 6 inches in diameter and you deducted from the loss observed at a given rate the loss with an empty tube at the same rate. I was wondering if

you would get the same results with a 1 inch tube or with a 12-inch tube.

Mr Hulbert: We have every confidence that we can get the same results.

Mr. Dixon: I just raise that because in some hydraulic experiments the results are difficult to explain.

Mr. Hulbert: We have a battery of $1\frac{3}{4}$ inch tubes for filtration experiments and we promptly found that those were not suitable and workable for this purpose, due to the fact that the water passing through the small tubes slowly changed appreciably in temperature. The change in temperature liberates dissolved oxygen causing rapid plugging of the sand layer with air.

We were making an effort to determine the rate of head loss in different kinds and depths and grades of sand with a given applied water. Consequently if the rate of head loss changed due to air liberation the data have no relation to the rate of head loss from the removal of coagulated matter. For that reason we switched from the small to the large tube designed for filtration experiments. We used the larger tubes for this preliminary study with the hope of providing ourselves with some foundation in the pure hydraulics of filter sands, before we attacked the much more complicated and difficult problem of filter clogging rate.

I have every confidence that the hydraulic data based on the large 6-inch filters are fully dependable and accurate. If they had not been we would not have gotten such beautiful plottings on curves and straight lines that you saw here this afternoon. Certainly if there had been any wall effect or arching of the sand in the tube it would have shown up in some of those sands and we would have found inconsistencies in the plotting, which we did not get.

Daniel E. Davis (Pittsburgh, Pa.): I heard the authors make mention of Slichter's work years ago, particularly referring to wells. Some years ago there was a problem that engaged our attention in the yield of wells in two entirely different districts in a certain community. Fortunately samples of the sand from these wells had been saved and the logs of the results of the well yields at the time when initial tests were run. We made sieve analyses of these different sands from each

well, and found that so far as the question of sand size was concerned, (using, as I recall, the effective size as the basis for determining sand size), the well yields did correspond to Slichter's thought that they would yield as the square of the sand sizes. In that case, however, we assumed that the porosity would be identical in each case, and that the sand size was rather an interesting development that accounted for the difference in yields in the two districts.

Philip Burgess (Columbus, Ohio): Some years ago I spent a year in testing sands. I have never at any time read anything that was presented as well as the subject was presented today. The authors of that paper are to be congratulated.

There is one thought, however, that appeals to me and I wish to ask a question concerning it. Did you rate your sieves, and if you did not I will call your attention to the fact that you may have overlooked one source of difficulty?

I have found in my experience if you want to get an absolute separation of the particular sand you must rate your sieve with that sand. I have found through actual experience that two different sands can give on the same sieve a difference of separation as high as 20 per cent. The reason for that is this: your sand may be a crushed material and the longer particles will go through the sieve more readily than the round particles.

I think if you would relate your sieves for the fine material you would find there may be a difference. The fine material, as you say, is made up of different shapes. If you have used different kinds of sand and applied the same separations of your sieves I would suggest that it is extremely important to test your sieves and see if the separations are correct for all the sands you use.

WATER RATE STRUCTURES

By R. E. McDonnell

(Of Burns and McDonnell Engineering Company, Kansas City, Mo.)

Municipalities and water companies usually call in an experienced engineer for making studies and investigations of needed additional storage, reservoirs, water supply facilities and general water works improvements. Engineers specializing in these problems are in the best position to render competent advice and assistance in developing the water works program.

When it comes to the question of establishing a water rate, however, the average layman and city official feels he is thoroughly competent. As a result of this lack of care and study in establishing water rates, in nearly every city we have water rates that are properly subject to severe criticism as unfair and unbalanced. In some cases the rates to certain classes of users are too high, and in others too low; and, in many instances, the rates to some of the large users are inadequate to cover the actual cost of supplying the water.

Water works plants are never completed, for there is a constant extension, enlargement and improvement to the whole water system. Consequently, there is frequent demand and occasion for revision and adjustment of rates upward or downward.

The schedule of rates for light and power often received serious study and consideration by engineering experts, but, it is a rare thing for municipalities to make any scientific study as a basis for establishing water rates.

It is not easy for local engineers and elective officials to make an unbiased, impersonal presentation of the facts, conclusions and recommendations regarding rates; and it is often difficult to correct irregularities and inequalities in fairness to all water users. Many municipalities have encountered violent opposition to a rate adjustment program, and there is no feature of the municipal business that will provoke so much discussion and argument as one of rate revision, especially if it becomes necessary to raise the rates.

Water superintendents, water boards and city officials would save

themselves much trouble and annoyance by having an engineer thoroughly familiar with rate problems make for them an unbiased and impartial rate study and investigation. Its adoption and approval, when based upon a complete engineering and business analysis of the water business, would meet with only slight opposition.

To permit an unfair or unbalanced water rate to continue only brings criticism upon the city officials and a constant feeling of unfairness on the part of water users. To purchase any commodity at a rate different from that enjoyed by your neighbor always brings criticism; and, many water works departments are unpopular with their customers because of a real or fancied unfairness in water rates.

Water rates should be based upon a careful study over a period of years of the revenue, income, depreciation, and the increasing amount of funds needed for replacements and constantly increasing operating costs. These items, being variable and changeable, make a rate revision necessary at intervals. Any changes in the water department, such as the inauguration of filtration, water softening, iron removal or changes in pumping equipment, often make it advisable to change rates.

A study of the past operating costs and revenues is essential for the determining of rates of the future. The basis for rates for the future must be the expenditures and obligations of the past and present, and the best possible estimates of future costs and obligations. These obligations include everything necessary to operate and maintain a water system properly, and to continue to furnish good, pure water in ample quantities.

The rates for the service rendered by the municipal works, for or on account of which any indebtedness is created, should be so fixed as to provide for payment at maturity of the principal and interest of such indebtedness, in addition to all other obligations and liabilities payable from the revenue funds pertaining to such works.

FUNDAMENTAL RATE CONSIDERATIONS

The operating cost and depreciation figures are usually available in most water departments, but occasionally a revision is necessary of the depreciation figures which local superintendents are using.

When public utility rates are fixed or modified, certain fundamental considerations must always be kept in mind. These considerations, with special reference to the water service, may be expressed in this way:

1. The water rates, or the income of the department from all sources, must meet the total cost of the water service.
2. This total cost is divided into
   (a) Adequate operating expenses under reasonably efficient and economical operation
   (b) Depreciation
   (c) Taxes
   (d) Interest on water bonds
   (e) Amortization of water bonds.
3. When the actual total cost of the service is ascertained, the burden of this cost must be spread as fairly and equitably as possible over the different classes of water users.
4. The determination of the rates for the different classes of service should be based
   (a) On the cost of the particular class and character of service
   (b) On considerations of the city's economic policy.

REASONS FOR RATE CHANGES

The most common cause of rate revision is the discovery of an operating deficit, which should, of course, be remedied promptly. Rate revisions have frequently followed the installation of water purification plants, softening and iron removal installations. A municipality can hardly inaugurate major improvements of this kind without adding to the revenues, for the operating expenses increase by reason of the installation and operation of purification plants.

The location of new industrial plants, which will be large users of water, often calls for a revision of rates.

Cities enjoying the increased economy of operation through Diesel oil engine power have found it possible to revise rates downward. The change from antiquated steam plants to electric power has, in some cases, required downward revision, but, in some cases, upward, because of the increased operating expense.

Change of source of supply and the inauguration of major improvements frequently demand an adjustment of rates.

The accumulation of too large a surplus cash fund is occasionally the cause of a rate investigation. This is an uncommon occurrence, but it actually was the cause of the rate revision inaugurated by the City Manager of Cincinnati, Ohio.

The constantly increasing difficulty of voting bonds for needed improvements has caused water departments throughout the country to realize more than ever before the necessity of accumulating a reserve fund for ordinary extensions, enlargements and improvements

out of the revenues, rather than from bond issues. Through failure of bond issues many cities have been compelled to revise rates in order that the future operation, extension and enlargement of the water plant would not be embarrassed.

Rates unreasonably low and barely sufficient to pay the ordinary operating costs, are not possible for the building up a depreciation reserve cash fund for major improvements and replacements. Such low rates are not creditable to any water department.

It is not advisable to increase rates for the providing of new capital that should properly come from bond money, especially on the major improvements.

Any proposed rate adjustment should be a fundamental measure enabling the water department to be placed on a sound financial basis, with sufficient income to meet all obligations and liabilities from the revenues pertaining to such works.

An item that has drawn more heavily upon the revenues in water departments than any other is the construction work and operating expenses required to maintain the purity of water. This often requires separate low service pumping stations, the construction of water purification systems, reconstruction of many reservoirs, providing roofs for uncovered reservoirs, and, in many ways, adding to the burden of operating cost.

WATER RATES OUTSIDE THE CITY

Some municipal water plants supply users outside the city at the same rate as those inside the city. This frequently brings criticism upon the water department management, the contention being that the citizens inside the city limits are taxed for paying bonds, which those outside the city escape. For this reason there should be a higher rate to users outside the city.

The cost of maintenance of distribution lines outside the city is usually higher than inside, because of the difficulty of protecting mains along unpaved and ungraded roads.

The general policy in most municipalities is to apply a 50 percent higher rate for water service outside the city limits, and there seems to be some justification for about this amount of increased charge.

COMPARISON OF RATES WITH OTHER CITIES

The common practise of municipalities in establishing water rates is to compare their rates with those of other cities. This is always

of interest, especially to the citizens, but the conditions upon which the rates should be based are rarely the same, and such a method of establishing rates almost always leads to criticism. One city may have to go twice as far for its water supply as another. Gravity water may be possible with one city, while in another the topography would be such that two, three, four or more pumpings will be required.

In Los Angeles there was a difference of 1400 feet between the low and the high levels of the municipality. This requires high pressures, pumping plants, reservoirs, different pressure zones, and conditions that greatly add to the operating cost.

The extremes of pressure existing in the distribution system in Los Angeles ran from 25 to 340 pounds per square inch. This range of pressure is about three times greater than in any other large city in the country, but it was not thought advisable to vary the rate on a basis of anything except quantity of water.

The policy of serving high and low elevations and long and short distances at the same rates has become so universal throughout the country that it would seem unwise to establish any different rate in the same city for this service. The water user has a right to expect pure water at adequate pressure at the same rates as any other user in the same municipality, the only variation of rate being because of the quantity used.

In comparing the water rates with other cities the chief interest for the laymen and citizens is in knowing if they are charged more than their neighboring city for the same commodity, and under the same conditions. The rates of other cities should not be used as an argument for establishing higher or lower rates, but it is always of such interest that any rate study and investigation should incorporate in it a comparison of a number of cities similarly situated.

In these comparisons it will usually be found that cities operating under similar conditions have invariably similar rates.

A summarized average is shown in table 1 of the cost of water in 100 cities under 25,000 population, 50 cities between 25,000 and 50,000 population, 25 cities between 50,000 and 100,000 population, an average cost of water in 225 municipal plants, and the average cost of water in 25 private plants. Table 1 is quoted from the booklet entitled "Rates, Revenues and Results of Municipal Water Works in the U. S." issued by my firm.

One difficulty of comparing water rates of one city with another is the great difference now existing as to how the extension of mains is

paid for. Some water main extensions, betterments and enlargements are paid for out of rates, while others are paid for by taxation or assessments against the abutting property.

The water consumption per capita is always an important factor in arriving at fair and adequate rates. If the water consumption is unreasonably high, a correction of this condition should be made before any establishment of rates.

WATER CONSUMPTION IN RATE MAKING

The writer was once called in to examine and pass upon a rate schedule in a Western city where it was found that the water consumption was 400 gallons per capita per day. The city officials were promptly advised that it was not a rate expert they needed but a meter salesman. Because the illegitimate use of water was not curtailed, the pumping capacity was inadequate, the reservoir capacity inadequate, and the entire plant had difficulty in supplying sufficient water.

TABLE 1

Average cost of water delivered to consumer in cities with municipal water works

	666 CU. FT. OR 5,000 GALLONS PER MONTH	1,332 CU. FT. OR 10,000 GALLONS PER MONTH	6,666 CU. FT. OR 50,000 GALLONS PER MONTH	13,332 CU. FT. OR 100,000 GALLONS PER MONTH	66,666 CU. FT. OR 500,000 GALLONS PER MONTH	133,332 CU. FT. OR 1,000,000 GALLONS PER MONTH
100 cities under 25,000 population	$1.54	$2.79	$10.88	$18.72	$67.64	122.47
50 cities between 25,000 and 50,000	1.48	2.74	13.64	19.72	73.69	133.47
25 cities between 50,000 and 100,000	1.26	2.14	8.79	16.17	60.87	107.11
50 cities 100,000 population and over	1.12	1.94	8.64	15.97	65.36	119.30
Average cost in all 225 municipal plants	1.35	2.40	10.49	17.64	66.89	120.59
Average cost in 25 private plants	2.24	4.00	15.74	27.77	97.68	148.53
Per cent higher for private plants	65.92	66.66	50.04	57.42	46.00	23.16

The water consumption per capita will frequently be four times as high in an unmetered city as in one with complete metering.

The adoption of a universal system of metering of water has, undoubtedly, saved millions of dollars to the citizens by the elimination of waste; and it is hardly possible to establish any fair schedule of water rates without first establishing a meter system. The water rates of an unmetered city will frequently be approximately double those of a city with a universal meter system. In a comparison of the water rates throughout the country it is clearly shown that the city with the highest number of meters has the lowest water rates; and, yet, the experience has been that the metering has not curtailed the legitimate use of water.

LOWER RATES TO LARGE USERS

Complaints frequently arise because of the very low rates that are made to large users of water, and, occasionally, because of the low rates made to the smaller adjoining municipalities which buy water wholesale from the municipality.

Wholesaling of a commodity of any kind can be done at a lower rate than retailing. If one water user through one meter consumes a million gallons of water per day it can be readily seen that this large user of water can be served at a relatively low cost compared to a million gallons of water per day being delivered to approximately 10,000 users; and the keeping of the records, billing, meter reading, etc., entails a very large amount of expense and clerical work that does not exist with the wholesale user of water.

The large users make it possible to build up a good load factor and if it were not for the large users, even at their low rates, the smaller users would be compelled to pay a higher price for water. In other words, his water cost is lower than it would otherwise be, if it were not for the large users.

Rates to these large users can, therefore, be made low with justification, but, as sometimes exists, they should not be below cost, for there should be a reasonable profit on all users.

These large users and industries are undoubtedly of value to the community, but free water or water below cost to them is unfair to the other water users.

It has become necessary in a number of cities where water supplies are easily available from wells, to make rates to large users extremely low and just slightly above the cost of production, and thus prevent the industry from installing its own water plant.

MINIMUM RATES

It is universally conceded in water works management that a minimum rate should be established for all users at least sufficient to cover the fixed charges of maintaining a service, which should be an amount ample for maintaining the records and services of the small users.

Every city, both large and small, has approximately 5 to 10 per cent of these small users to whom a minimum bill should apply.

There now exists a great variation in the minimum amount that is charged in various municipalities, ranging from $0.50 to $2.00 per month, in some of the Western cities. A minimum bill of $1.00 per month seems to be an average charge for water service to small users.

SOURCE AND ANALYSIS OF INCOME

The principal sources of income are several and each source of income is intended to yield sufficient revenue to pay for that particular class or character of service, making every division at least self-supporting, with a rate based upon the cost of operation and the capital invested. The rates should be commensurate with the service rendered, and not unbalanced with one class of consumers paying too high a rate because another class pays too low a rate. If one class of user pays a rate too low then some other classes must pay a higher rate than would otherwise be necessary in order to provide sufficient income. A rate fair to all, with large and small users paying their just proportion of operating costs, fixed charges and capital invested, is the primary objective of a disinterested and unbiased rate investigation.

It is always easier to let conditions remain undisturbed, but when your consulting engineers are faced with the need of increased storage reservoir capacity, larger mains, additional sources of water supply and knowing the need of large replacements, it is necessary to inquire into the adequacy of the present rates yielding a revenue to do these necessary things.

The sources of income are:

(a) Water sold and used for irrigation purposes
(b) Water used for municipal uses in fire hydrants, parks, sewers and street flushing
(c) Income from sprinkler services in building
(d) Income from minimum charge for all meters

(e) Income from installation of consumers' services and street main extensions
(f) Income from land rentals, farms and other sources
(g) Income from domestic water sales on a meter basis.

An analysis of the total gross income from all sources frequently discloses the information that the water system is not yielding an adequate return to meet the necessary expenses.

These expenses or necessary demands on income consist principally of the following items:

(a) Source of supply, such as aqueducts, tunnels, dams, siphons, reservoirs, and all headworks development
(b) Pumping stations
(c) Purification works
(d) Transmission power lines for water requirements and distribution system expense
(e) Commercial expenses, such as billing and collecting;
(f) General expenses
(g) All taxes of every nature
(h) Interest on funded debt
(i) Depreciation
(j) Other expenses.

WATER FOR MUNICIPAL USES, PARKS, SEWERS AND STREET FLUSHING

This is a yearly indirect and uncertain income in many cities, but it is a recognized legitimate charge if the water department is to be made self-sustaining. Many water departments make a direct monthly charge to the fire department for each hydrant; and the street department in many cities is charged a sewer flushing charge at a price per jet for the sewer flushing. Water departments should be paid an amount for street flushing, also for there is no reason why the water department should contribute free water toward flushing of streets to some other department without the proper charge.

The entire community derives a benefit from adequate fire protection, and its cost should be charged to the whole population. The only fair and equitable method of getting the revenue for the fire protection furnished by the water department and placing it where it belongs, is for the individual tax payer to pay for fire protection in proportion to the value of the property protected—not according to the amount of domestic water he uses, or the fixtures in his house.

When a proper charge for fire protection has been found it should be collected in a general tax, each taxpayer paying according to his assessed valuation.

Fire protection costs more to the city than merely the furnishing and maintaining of fire hydrants. In order to maintain good fire protection, pumps, mains, reservoirs and the entire water system must be larger than it would otherwise need be for domestic water and other ordinary uses.

Fire demands often come at the peak hours of domestic consumption. This ready-for-service cost can justly be met only by water rates based upon the increased investment necessary because of fire protection needs. The principal factor in determining the size of a water main is its fire stream capacity, and not its domestic water capacity.

In many cities investigations show that 20 to 30 percent of the total cost of a water plant is due and chargeable to its preparedness for fire protection.

The per-hydrant-charge is merely a convenient means of arriving at a measure of rate and collection. The hydrant is not the measure of benefit or use. In Indianapolis the U. S. Supreme Court held that $1.31 per capita would be a fair charge for fire protection. The Wisconsin Railroad Commission has established for large cities $1.25 per capita as a fair rate on the per capita basis.

A list of fifty cities with populations of from 10,000 to 15,000, selected at random, shows that these cities maintain a total of 7800 fire hydrants or an average of 156 hydrants per city, or 8 per 1,000 population. The smallest number of hydrants maintained by any of these cities is 25 hydrants, while another city maintains 401, or more than 16 times as many.

The rentals vary from a low of $19.00 to a high of $100.00 per hydrant, with an average of $47.00. Computing on this basis, we find that the average cost of fire protection or rather fire hydrant rental to these cities is 156 hydrants at $47.00 per hydrant or $7332.00. This amount, of course, is raised by taxation. In addition to this there are other charges for sewer flushing and street washing. In addition to paying the water rates asked by the company the citizens also have to pay for the fire protection and other city water needs through the medium of taxes.

In most cases where the water system is municipally-owned, the hydrant rental, street washing and sewer flushing is furnished free to the city and it has been found that even with lower rates than those in effect under private ownership, the municipal water system

is able to pay all operating expenses, furnish the city its needs without charge and still show a nice profit.

INCOME FROM SPRINKLER SERVICES

The installation of automatic fire sprinkler services in warehouses, garages, factories and mercantile establishments provides excellent fire protection at a minimum of expense.

The saving in insurance rates is considerable because of the sprinkler service. This amounts to from 50 to 90 percent of the insurance premiums. In some cases the sprinkler service will pay for itself in from one to three years. The usual plan is for the owner to install at his own expense the entire service, and the water department to furnish the water at nominal rates sufficient to maintain the service.

The water department and anyone familiar with the sprinkler service realizes its value from the fire protection standpoint, and no one wants to place a rate upon it that would lessen its use. On the other hand, the sprinkler service is made valuable to the owner mainly because of its preparedness by reason of mains, pumps and reservoirs, outside of the sprinkler equipment itself; and a moderate charge and revision of rates is necessary in most cities.

PROCEDURE IN RATE MAKING

Whenever a schedule or ordinance is revised, the opportunity should be taken to eliminate any unbalanced rates, and to make the charge for the different classes of consumers as nearly equitable as possible, so that each one should contribute his proper share of cost in proportion to the benefits received.

The first factor in revising a rate schedule is to make certain that the consumer will pay a bill which is reasonable from the standpoint of the cost of the services; that is, the small and the large consumer, the manufacturer, the apartment house owner and the property owners in general, should pay in proportion to the benefits received. Under no circumstances should water be supplied below cost. When water is sold at less than cost it means that this loss is being carried or made up by the revenues derived from some other consumer. A rate unduly low for one must mean a rate unduly high for others.

A water works plant is never completed. As the city grows in population, the entire system must grow in capacity. As the city expands in area, the water works must extend in its distribution system. To supply water as required, the revenue produced should

be sufficient to pay all operating costs, interest on outstanding bonds, and to provide for all needed replacements, extensions and betterments commensurate with the growth of the city.

The rate should be so fixed as to produce as closely as possible average requirements for a reasonable period—say, from 5 to 10 years in the future—and yield sufficient money to care for a betterment program over a period of 8 to 10 years. It is always easier and pleasanter to lower rates than to raise them; and, if unexpected favorable conditions should produce more revenue than necessary, a downward rate adjustment is quickly made.

It is a well-established principle, universally recognized by courts and commissions, that the users of service (either private or public) should pay for the service in proportion to the costs they entail, and the value of the service rendered; that public service, such as fire protection, sewer flushing, streets, parks and public building service, should be paid for by the city, and collected in the general tax. Such service is for all the people and the most equitable way of securing its cost is by assessment in taxes which distributes the cost in proportion to the value of the service.

REVENUES OF FIVE CLASSES

If the water works property is to be wholly self-supporting it should earn:

(1) Operating expenses of all kinds;

(2) Interest upon the investment or debt incurred;

(3) An annual allowance for depreciation which should be equivalent in cash or marketable securities in possession of the public. These items are clearly cost of service.

(4) Annual contributions from revenue to a reserve or surplus fund to care for emergencies and the unexpected expense necessary in any fast growing city, avoiding serious and costly delays while waiting for bond elections and bond issues, and possibly making bond issues for minor capital expenses unnecessary.

(5) Annual contributions to a sinking fund to retire bond issues as they become due. The earnings of a municipal plant should be sufficient to create a fund to retire the debt. This charge, however, cannot be construed as a direct cost of service. The schedule of rates designed is intended to furnish earnings to cover the first four of the above items.

The thought often exists in the minds of the laymen that all

revenue should be derived from water sales. This is not sound, when one considers the valuable vacant property or the large office buildings which enjoy water service, fire protection and increased property value out of all proportion to the relatively small charges for water used.

Fire protection service is rendered to property or to property owners in proportion to the value of the property protected. Charges for such service, therefore, should be entirely against the property, or against the general tax levy. In fairness to all concerned, a charge for fire protection must appear separate in any schedule of equitable rates. The provision of a water supply for fire protection service, whether by use of fire hydrants or sprinkler systems, creates an increase in plant investment and in plant operation over and above the costs which would be sufficient for general water service.

The total annual revenue to be raised to meet safely all the above requirements should be divided into three general classes:

(1) The revenue to be raised by charges to individual users.

(2) The revenue to be assessed against property by reason of its enhancing value because of improvements, extensions of mains, assessment districts, new services, etc.

(3) The revenue charged against the public because of the general benefits of the water works, such as fire protection, sanitation, beautification, health and safety.

Any contemplated rate revision, either up or down, should embody these general requirements, and a careful survey and study by a disinterested water works engineer is fully justified in time and expense.

(Presented before the Rocky Mountain Section Meeting, October 24, 1932.)

WATER RATES AND CONSTRUCTION POLICIES IN MUNICIPALLY OWNED PLANTS

By L. R. Howson

(Of Alvord, Burdick and Howson, Consulting Engineers, Chicago, Ill.)

The normal American water works is a growing institution. The growth of the water works and the expenditures required therefore in general parallel the growth of the community served. The records of the United States Census Bureau show that the normal American city doubles in population in from twenty to twenty-five years which represents an average rate of increase in population of approximately 3 per cent per year (compounded). The average value of water works properties at the present time is from $40 to $50 per capita served. The expansion of water works facilities to care for the growth in a normal community will require, therefore, additional expenditures for capital account on an average amounting to from $1.20 to $1.50 per year per capita served. This figure will be larger for communities of rapid growth and correspondingly less for slow growing cities. As the total income of the average water works at the present time is not far from $6.00 per capita, it is evident that the average financial requirements for new construction are from 20 to 25 per cent of the total income.

Every water works, whether municipally or privately owned, should be self-supporting and self-perpetuating. The rates must be adequate to support the present service and to permit orderly expansion to care for growth. Thus the close association of water rates and construction policies is apparent. In Ohio, the statutes specifically recognize the inter-relationship of revenues and construction expenditures by permitting municipalities to issue bonds for water works construction without such bonds being charged against the indebtedness of the City, provided the income from operations of the water plant is sufficient to carry the fixed charges and retirement expenses on the bonds.

Adjustments of the rate schedule, particularly if in an upward direction, are always attended with more or less discussion and con-

troversy prejudicial to the relations existing between the utility and its consumers, and it is therefore highly desirable that there be as few changes in the rate structure as practicable. This fact makes it increasingly important that the expenditures required for construction be distributed insofar as practicable so as to require only a minimum of rate changes. The first essential to a smooth distribution of construction expenditures is the formulation of a comprehensive construction policy covering the requirements for both the near and more distant future.

CONSTRUCTION POLICY

Fortunately, it is entirely practicable in most cases to so develop a construction policy and program as to provide for the orderly expansion of the water works without placing a burden either upon the present or future consumers. Such a policy involves a careful forecase of future requirements, reviewed periodically in the light of actual conditions as they mature and adjusted in conformity therewith. Water requirements in general approximately parallel population growth. It is, therefore, practicable with reasonable accuracy through a forecast of population growth, to forecast the future water requirements, study the various practicable methods of meeting them and develop a comprehensive plan for progressive enlargement of the various parts of the water works such as the pumping and purification facilities, distribution system, feeder mains, etc. The approximate order of construction of each step can be determined, the cost estimated and the total expenditures so distributed as to effect a relatively uniform demand for funds. This can be practicably laid out for a generation or more in the future. Such a program, should, of course, be reviewed at intervals of approximately five years and comprehensively adjusted to fit the matured conditions at not to exceed 10 year intervals.

Such a comprehensive survey of existing facilities and their deficiencies for the present and prospective requirements accomplishes the following important objectives:

1. It assists in avoiding excessive obsolescence.
2. It provides facilities just prior to the time when they are required and thus saves the fixed charges on expenditures made before necessary.
3. It assists toward the most practicable uniformity of construction requirements and expenditures.

The first two accomplishments are reflected directly in the water rates which must be collected from the users who must pay for the losses due to both obsolescence and extravagance. The third is reflected in the degree of satisfactory relations between the utility and the users of the service.

FINANCIAL POLICY

If the development of a comprehensive construction policy is practicable, it should follow that the financing of such a policy is equally practicable. The two usual methods of financing municipal water works expenditures are from (1) bond issues; (2) current income.

Both of these plans are practicable and each has its advantages and disadvantages.

Anyone who has had extensive contact with municipal water works construction projects is thoroughly familiar with the difficulties incident to the passage of bond issues for construction projects at the time when they are needed. The present depression has witnessed the defeat of many well matured projects, the financing of which was dependent upon the passage of a bond election. Another serious drawback to the raising of construction funds by bond election is the fact that so frequently the issuance of bonds is decided upon purely political or other lines entirely foreign to the necessity for the improvement, frequently resulting in the failure of meritorious projects and somewhat less frequently in the passage of ill advised construction projects. In the wake of a hard fought election over a water works bond issue, there is left a trail of dissatisfaction from those who were in the minority.

The policy of financing construction expenditures from current earnings has the following advantages:

1. It obviates delays incident to the failure of bond issues.
2. If the rates and construction policy are carefully coördinated, it provides the funds when needed.
3. It distributes most uniformly, the construction expenditures and therefore, equalizes the payments by the consumers. Construction from current earnings obviates the bulges in fixed charges incident to the floatation of bond issues.
4. It does not periodically focus the attention of the users upon *costs* rather than *service*.

The principal objection to financing water works improvements

from current earnings is that the existence of a water works fund even though specifically set aside in reserve for construction purposes, is a temptation to diversion of the water works revenues to the support of other City Departments.

Financially considered over a long term, the two policies of financing construction expenditures, namely, by bond issues and by current earnings, are equal in cost to the consumer. In the former case, the water works borrows the money, the consumer pays his share of the interest on it and from time to time, his share of the amortization of the indebtedness. In the latter case, he pays outright and the transaction is closed. As before stated, over a long period, the cost to the consumer is exactly the same.

The cost to the consumer of financing utility construction from rates is relatively small amounting to from $\frac{1}{3}$ to $\frac{1}{2}$ cent per capita per day. Since the consumer must pay the same cost of water service whether the financing is direct from revenues or indirect through bond issues, why should he not be given the assurance of adequate service expanded as needed which can best be done by financing from the current rates.

A number of cities in Wisconsin as elsewhere have been proceeding quite largely along the line of financing construction expenditures from current earnings. Others have supplemented this method with bond issues of short duration. The Milwaukee water works with one of the lowest water rates in the country, has practically no bonds outstanding. Racine and Kenosha for a period of ten or more years have financed their construction almost entirely from current earnings.

The experience of Racine is rather remarkable when it is considered that the City acquired the property only thirteen years ago and in that time, has reduced the purchase indebtedness from $1,225,000 to $976,000 (20 percent) and at the same time, has built a new intake, filter plant, elevated storage, pumping station, nearly 60 miles of mains, etc. at a cost of over $1,900,000 financed almost entirely from earnings. All of this construction was executed according to a carefully prepared comprehensive survey made soon after the property was acquired and reviewed periodically since. During all of this period, the service has been continuously improved. This program has been executed to the very general satisfaction of the consumers even though the average construction expenditure at Racine due to the underbuilt condition of the water works when taken over by the

City, has been approximately $3.00 per year per capita or about double that normally required.

Kenosha has also been following a carefully outlined construction policy for nearly ten years and in that time, has provided a large clear water reservoir, enlarged the filter plant, increased both high and low lift pumping capacity and added approximately 30 percent to the mileage of mains, all financed from current earnings. The outstanding bonds on the water works have also been reduced about $25,000 per year during this period. Kenosha has financed new construction and retirements averaging approximately $2.50 per capita per year from current revenues, with general satisfaction to the community served.

These specific illustrations are given to show that what the consuming public really wants is good service. So long as the service is good, the rates reasonable and the consumers not periodically confused by misrepresentations which accompany most contested bond elections, there is little complaint from them.

The rate schedule may be made sufficiently elastic to permit the current earnings to cover the variation in construction expenditure requirements in several ways. Possibly the simplest is through the adoption of a basic rate schedule adequate for the most severe construction requirements to which a variable discount rate is applied. In periods when the construction requirements are less than normal, the discount rate for prompt payment of bills may be increased and when construction requirements are above normal, the discount correspondingly reduced. At Louisville, Ky., several million dollars of construction have been financed in the last few years wholly from current earnings with no change in the basic schedule, but with the discount varying from 10 to 33 percent.

If any financing plan is to work out equitably, it must be applied on a businesslike basis, which requires that:

1. The water works should receive pay for all services rendered; and
2. The water works should pay for all services which it secures.

There is no such thing as "free water" or free water service. It is simply a question of shifting the cost from one consumer to another. In many municipalities, there is no payment by the City to the Water Department for fire protection, nor does the utility pay to the City anything in the way of taxes.

The failure to collect for fire protection through taxes is in effect

an inequitable distribution of water works operating costs. When it is considered that in the normal water works, the large consumers, constituting less than one-half of one percent of the total, ordinarily use about 40 percent of the water sold and that these largest consumers normally include the railroads, and other large users of boiler water, it is evident that fire protection paid for through domestic rates is not equitably distributed.

Taxes on water works properties normally range from 1 to 1½ percent of the fair value. The equitable allocation of total income as between fire protection and water sales normally results in from 10 percent of the gross revenue for large cities to 30 to 40 percent of the gross revenue for cities of 25,000 population being assessed against the fire protection service. This is equivalent to from 1 to 3 percent of the value of the property. So far as the dollars and cents of the transaction is concerned, when taxes are balanced against fire protection charges for cities of approximately 300,000 people more or less, there is little inequity. However, for a city of 25,000 people, an equitable charge for fire protection service would normally result in an annual charge equivalent to approximately 3 percent of the value of the water works plant or from two to three times as great as an equitable assessment of taxes. In the smaller communities, therefore, the balancing of taxes against payment for fire protection results in inequity to the Water Department.

ADMINISTRATION

Adequate correlation of operations and construction can only be practicably secured by continuity of administration. No program can be laid down and consistently followed under frequent and periodic changes in management. Appreciation of this fact has led to the creation of Water Boards and Water Commissions in many of the municipally owned plants. Such a Commission ordinarily consists of from three to five men selected from the outstanding men of the community. The members should have overlapping terms so that there is always a majority of the Commission experienced in the administration of the water works affairs. Appointments to fill vacancies are usually made by the Mayor with the approval of the Council.

The advantages of water works administration by such a Commission as compared to the Aldermanic Council administration is obvious. The Water Commission has nothing to consider other than the

administration of the water works. With the members of the City Council, the affairs of the Water Department are incidental. The personnel of the City Council is subject to all the influences incident to frequent elections and short terms. All too frequently, City Councils administer water works along the lines of expediency rather than along the lines of efficiency. There are, of course, many notable exceptions to this general statement, but there is a quite definite tendency in municipal plant operation toward placing the municipal water works under a non-political overlapping term Commission.

This discussion may be briefly summarized as follows:

1. The water works in a normal American city is a growing institution ordinarily requiring an expenditure for new construction equivalent to from 20 to 40 percent of the gross income from operations.
2. Municipal water works as well as private should be self-supporting. The funds for construction as well as operation must, therefore, ultimately come from the income of the property.
3. It makes no difference to the consumer in dollars whether he pays for the expansion of the water works directly from current earnings or as interest and retirements on bond issues.
4. Every water works should have a comprehensive construction policy outlined for years in advance. With such a policy, it is practicable to distribute construction expenditures relatively uniformly.
5. Relatively uniform distribution of construction expenditures makes practicable the financing from current earnings. This in turn offers the most practicable method of having funds available when needed, of avoiding delays and complications arising from bond issues and of even greater importance, offers the method of financing most acceptable to the consumers themselves.
6. Successful development of a construction policy requires continuity of administration which can usually be best secured for municipal plants through a Water Commission, the members of which have moderately long overlapping terms and which is so created and constituted as to be as well removed from politics as possible.

The essentials of successful water works management involve a continuing policy, a well developed program of betterments and a rate structure adequate to finance them.

(Presented before the Wisconsin Section meeting, October 12, 1932.)

WATER, CHEAPER THAN DIRT

By Charles A. Haskins

(*Consulting Engineer, Kansas City, Mo.*)

Whenever the financial considerations of waterworks are being discussed, particularly along with those of the other public utilities, it is almost certain to be remarked that water-works operation does not pay. Statements to this effect have been made so often, the impression is widely spread that this most essential utility cannot be, or at least is not generally, operated profitably. Of course, it is realized even by those who believe this fantasy that the water-works system is indispensable to a populous community, and that there are many benefits derived from the establishment and operation of an enterprise of this character other than those of a pecuniary nature.

Perhaps this opinion, which actually amounts to a belief by many that the profits in water-works operation are more of a social than of a financial nature, is responsible for the fact that more than three-fourths of all municipal water-works are publicly owned, and they supply perhaps more than 90 per cent of the product, as compared with the municipal electric light and power industry, in which ownership is about equally divided, but with the privately owned systems supplying about 95 percent of the product. On the other hand, possibly the reverse is true, because the preponderance of municipally owned water plants, many with poor financial management and operated under loose business methods, may be responsible for the belief that water-works operation is not profitable.

WHAT IS PROFITABLE OPERATION?

Just what is meant by profitable operation? For the privately owned works it certainly means the earning of sufficient revenue for the payment of all costs of operation and investment, and in addition, a reasonable return on the value of the system for dividends on the money invested. The investment charges must include allowances for depreciation and retirement of the equipment so that the actual value of the equipment plus the allowances will be equal to and will

maintain the integrity of the investment. A small amount is usually set aside from earnings for the building up of a reasonable surplus to care for variations in earnings occurring in periods of business depression and to insure the payment of dividends at all times. On account of the fact that private companies are seldom if ever permitted to amortize the investment out of earnings, or to pay for the cost of extensions or improvements out of money secured from rate payers, it is essential that they earn a sufficient amount to maintain good credit in order that capital may be attracted when necessary.

There are differences of opinion as to what constitutes profitable operation for the publicly owned systems. Practically all such systems are financed by general obligation bonds and usually it is required by law that such bonds be paid within their term, 20 to 30 years, logically but not necessarily out of earnings. Private companies, on the other hand, are not permitted to pay capital expenditures out of operating revenue. As a result of this policy the municipally owned system generally is burdened with higher fixed costs in the first few years after its establishment, but later on it becomes possessed of a property with a considerable portion of the original value practically unimpaired because of the fact that the useful life of most of the structures is much longer than the period allowed for the amortization of their cost. This means of course that they then have little or no interest to pay on the original investment, and actually need only earn the relatively low depreciation and retirement allowances above the operating expenses, in order to maintain the integrity of the investment. When that time arrives the municipally owned plant should be in a position to pay all of its proper costs with exceptionally low rates for its output, and perhaps at the same time to return to the city at large much of the cash or credits which may have been advanced to make up deficits in the early days of operation, and also to pay to the city the accumulated taxes, at least the amount which the city has lost through ownership of the system.

Three policies are advocated for the operation of publicly owned utilities, and the degree of success attained should be judged by the rates charged and the income obtained as compared with that required under the policy selected. First is the self-supporting policy under which only the operating and fixed charges are required to be earned; second the self-supporting and the self-expanding policy under which the operating and fixed charges are required to be earned and in addition sufficient amounts to pay the cost of extensions and

improvements; and third either of the above policies and the earning of a profit as well. It is maintained by some that the payment of costs of extensions and improvements out of earnings results in the imposing of a burden upon the present consumer for the benefit of the consumer of the future. This is perhaps true, but if it can be done with reasonable rates, the policy has much practical justification, particularly in communities with slow growth or stationary population. On account of the fact that water-works systems are generally built with much greater capacity than is required for the consumer, not only in anticipation of an expanding population, but also for the purpose of fire protection service, the fixed charges of course are proportionately high. The demand of the individual consumers, or many of them, is almost negligible as compared with the fire demand, and the load factor is low. Therefore, usually many consumers may be added to a system without a corresponding increase in fixed charges, and the cost per unit of output thereby reduced, to the advantage of the consumer.

The operation of the municipally owned utility for profit seems an indefensible policy and one opposed to the fundamental theories justifying cooperative ownership, because it brings about an inequitable distribution of the tax burden. Then too there is the danger that a small profit will result in the demand for more profit, requiring either higher rates, which will in turn discourage consumption and increase unit costs, or the neglect of paying all of the proper charges, or otherwise tempering with proper management through political interference. There are some cities boasting of successful operation for profit, but investigation frequently shows either that their rates are not low, that all of the costs are not being paid or else that the bonded debt has been reduced to an amount far below the investment in the system and the required fixed costs are correspondingly low. This means that "operation at a profit" frequently is being accomplished at the expense either of the past or the future rate payers, or taxpayers, as the case may be. When the business reaches the state where rates are low and all of the present costs are being met and the past debts or credits have been repaid, and surpluses still occur beyond the reasonable prospective needs of the future, as no doubt can and does happen, then a portion of the surplus properly might be considered as profit and be devoted to non-productive public purposes.

There are several reasons why, in the opinion of the writer, the

water-works business whether privately or publicly owned will not yield the profit that can be expected from some of the other public utilities, particularly the electric light and power systems. While new uses and devices are constantly being developed which increase the use of electric current, the market for water, when all of the potential users have been reached, becomes more or less static and there is no incentive for greater consumption. Moreover the manufacturing costs of electric current generally decrease as the output increases while with water the production costs frequently increase with larger requirements, particularly for those places unfortunately situated as regards sources of supply. Water, being an essential commodity, is often required to be distributed at a relatively high fixed cost, into sparsely populated sections, often in advance of development, creating losses which may occur over long periods of time. Plans sometimes utilized to solve this difficulty consist either of charging the costs of water main extensions against the abutting or benefitted property or the requiring of guaranteed earnings on the investment (generally used by private companies), but there are many difficulties apt to arise in the application of either solution.

FIRE PROTECTION AND FREE WATER

The one great obstacle, however, occurs as the result of the high fixed charge on the portion of the system devoted to public use, that is, for fire protection, and the reluctance of governing bodies to assess the proper proportion of the cost to the beneficiary, the public, rather than to the rate payer. It is a classic statement among water-works men that one half of the entire cost of the system is made necessary by the fire protection requirements. This statement is probably true for a city of from 25,000 to 50,000 people, but it is generally less than this proportion in the larger cities and is considerably more in most smaller cities. Of course, the proportion cannot be definitely stated without a careful study of any particular case, on account of the varying conditions. Our own "Water-Works Practice" states, "The cost of fire protection service is approximately: (a) 25 to 35 percent of the fair gross revenue; (b) $2.00 per capita plus or minus 40 percent or about $1.25 per capita in large cities, $2.00 in average cities and $3.00 in smaller cities and towns." But does any one know of more than a very few cities where even approximately such amounts are actually allowed the water company or the department? The writer does not. If this cost is not paid by

the city at large through taxes, which seems to be the proper way to distribute its cost upon the beneficiary, then it must be paid by the consumer through rates, a manifestly unfair procedure.

Furthermore, in many cities, whether the system is privately or publicly owned, the public demands, and the water company or department gives, a large quantity of free water for flushing sewers, cleaning streets, for public buildings, parks, fountains, hospitals and often churches, and to private and public charitable institutions. It is often stated that the requirement of these donations is justified in the case of municipally owned systems because they are free from taxes and in the case of privately owned, that they are a return for the free use of streets. However, as in the case of fire protection service, the cost is too often borne by the rate payer rather than by the taxpayer, who of course is the true beneficiary. Many cities prefer to view the ownership of utilities as a governmental rather than as a proprietary function.

UNSATISFACTORY ACCOUNTING PRACTICE

Many cities and some private companies suffer from inadequate and incompetent accounting practices, and in fact sometimes countenance what might be termed dishonest procedure. In most states municipally owned systems are not subject to regulation by any competent outside commission with the result that the finances are often in a deplorable state. Except for the relatively enormous taxing power and wealth behind them many are in a hopelessly bankrupt condition. The present agitation against high taxes and for the self-liquidation of municipally owned projects of this character has given considerable impetus toward the often laudable practice of utilizing income or utility bonds for the financing of public works.

There are of course many water-works systems operating under sound financial policies, earning all of the costs of operation and investment, and yielding a reasonable surplus in addition, but there are also many which not only do not pay a direct financial return to their owners but which do not even pay the costs required under the minimum policy of self-support. It has been the observation of the writer that where such a condition exists it is usually due either to political or to extravagant management, which squanders and wastes, or to ignorant or simply to apathetic and indifferent management which perhaps unknowingly dissipates the assets through the failure

to institute an honest accounting practice, and an adequate and equitable rate schedule both for the commodity and the service rendered.

"WATER CHEAP"

Governing bodies generally consider necessary rate increases with great reluctance and they display frequently great hesitancy toward the institution of adequate payments for fire protection service and for free water, notwithstanding the fact that water is nearly always the cheapest commodity offered for sale. An expression is often

TABLE 1

Costs of some commodities per ton, delivered to consumer

COMMODITY	COST PER TON
	dollars
Water (to small consumer	0.072
Water (to large consumer)	0.032
Sand	0.70
Coal	6.00
Cement	10.50
Cast Iron Pipe (Kansas City)	43.15
Water-works Castings (6 in. and larger)	100.15
Lead	80.00
Gasolene	56.00
Milk	112.00
Bread	160.00
Meat (30¢ pound)	600.00

(After C. B. Hoover, Civil Eng., Aug., 1931)

heard "dirt cheap." A statement is presented in table 1, often used by the writer, showing the cost of water per ton, delivered to the consumer, as compared with current costs of some other commodities of common use. The cost of water as shown is based on the median rate charged for consumptions of 5,000 and 1,000,000 gallons per month as determined by a study of the schedules of the water works of the 25 largest cities in Kansas, 3 privately owned and 22 owned by the municipality.

Actually dirt, generally considered as the cheapest of materials, cannot even be excavated from a hole in the ground, under favorable conditions and with the most efficient machinery, for the price at which pure and wholesome water is collected, treated and delivered,

available instantly and at all times, in quantities large or small, to the consumer. The expression should be changed to "water cheap" because this, the most essential of all commodities, is actually far cheaper than dirt.

(Presented before the Missouri Valley Section meeting, October 26, 1932.)

STORAGE REQUIREMENTS FOR NORTHWESTERN STREAMS

By Richard G. Tyler

(*Dean, College of Engineering, University of Washington, Seattle, Wash.*)

The method developed by the late Allen Hazen for determining storage requirements of streams for varying drafts has not been widely used by other engineers and the present paper is to call attention to some of its advantages by presenting some preliminary data on streams in Washington and Oregon. The writer desires to present these data for the further reason that Hazen's estimates concerning the streams of these two states are too conservative. His method of study, however, calls attention to certain of the characteristics peculiar to stream flow in the Northwest concerning which little definite information has as yet been published.

The method referred to is described by Hazen in the Manual of the American Water Works Association on "Water Works Practice," pages 51 and 63 inclusive, and in the American Civil Engineers Handbook, 5th Edition, pages 1446 to 1452 inclusive. Briefly, it is a statistical study which consists of the determination of the *mean annual flow* of a stream for the period of record and the computation of a *coefficient of variation* to indicate the variability of the stream from the mean. These are the two most important characteristics of any stream, viz., its mean annual flow and the manner in which it varies from that mean.

The coefficient of variation is obtained in the following manner. Prepare a table of annual flows of the stream under consideration, using any convenient units, and find the mean. The difference between each annual flow and the mean is called its variation. These variations are squared and their sum obtained. The square root of the quotient of this sum divided by the number of years minus one, gives the *standard variation.* The *coefficient of variation* (c.v.) equals the standard variation divided by the mean annual flow. It will be noted that annual rather than monthly flows are used. Hazen's study showed that daily flows indicated a

required storage of about 9 days more than monthly flows. The annual means are satisfactory for c.v. determinations.

Hazen claims that greater accuracy can be derived for a stream where less than 20 years record is available, by considering its mean flow and c.v. than from a study of the mass diagram, and he gives tables in the Handbook above referred to, for use in making such studies. The method has the further advantage of making a more definite use of longer time records of adjacent streams through a comparison of coefficients of variation, than other methods now in use. These matters will be clarified as the discussion proceeds.

Certain streams in Washington were selected with the aid of E. I. Pease of the U. S. Engineers Office, which were thought to be typical of the different conditions encountered in the state. He also furnished mass diagrams for these streams which appreciably reduced the labor involved in this study. Other stream flow data were obtained from the Water Supply Papers of the U. S. Geological Survey and from Water Supply Bulletin Number 4 of the Division of Water Resources of the State of Washington. The number of streams studied in detail is small, but they show very clearly certain distinctive characteristics which will be pointed out. For convenience, the yearly flows of the streams included in this study are shown in table 1 and their means determined. It will be noted that there have been years of lower stream flow than 1930–31, but for many streams all over the country these were the most critical years. Similar data, with the exception of the mass diagrams, were studied for a number of other streams shown in table 2 and their mean flows and coefficients of variation determined. These, together with other miscellaneous data, are shown in the table. From the mass diagrams of the rivers in table 1 the reservoir capacities required to supply varying demands upon the stream were determined. For this purpose the 95 percent dry year was used, or that year than which 5 percent presented greater shortages of flow as indicated by their mass diagrams. Figure 1 was then prepared to show the relation between the required reservoir capacity and the draft, and for convenience in using these data, these were both expressed in terms of the mean annual flow. For example, the Skagit River would require a reservoir capacity of 0.535 times the mean annual flow of 4030 c.f.s. to supply a draft equal to 0.9 times this mean flow for the 95 percent dry year.

The dashed line in figure 1 is taken from the A. W. W. A. Manual and gives Hazen's data for c.v. = 0.20 for conditions of zero ground

storage. It will be noted that most of the curves fall below Hazen's curve, partly because of their smaller c.v., and partly their existing *ground storage*. This will be discussed in more detail at a later point.

TABLE 1

Yearly flows of streams included in this study

YEAR	MEAN DISCHARGE IN CUBIC FEET PER SECOND							
	1	2	3	4	5	6	7	8
	Skagit	Sauk	Baker River	Middle Fork Snoqualmie	Hungry Horse	Cedar River*	Chiwawa	Priest
1906						341		
1907						461		
1908						351		
1909	3,980					248		
1910	5,400					369		
1911	4,640		2,250			241		
1912	3,410		1,940	1,150		394		
1913	4,350		2,200	1,220		302		
1914	4,100		1,970	1,070	4,080	261	681	1,370
1915	2,750	3,720	1,660	720	3,050	182	531	1,340
1916	4,520	5,030	2,190	1,430	2,330	499	296	833
1917	3,690	4,170	1,910	1,200	4,420	428	673	1,280
1918	4,960	5,140	2,600	1,450	3,570	565	668	1,330
1919	4,680	5,270	1,980	1,230	3,160	442	521	983
1920	3,730	4,190	2,000	1,110	2,200	359	602	1,080
1921	5,140	5,180	2,350	1,390	2,970	522	493	1,130
1922	4,490	4,160	2,030	1,120	4,080	437	715	1,250
1923	4,170	3,910	1,790	1,120	2,660	456	441	902
1924	3,680	3,770	1,830	1,070	2,880	368	601	1,020
1925	4,770	4,860	2,210	1,170	2,930	471	477	775
1926	2,470	2,970	1,660	860	4,040	295	564	1,050
1927	3,930	4,740	1,890	1,320	2,650	408	341	802
1928	4,335	4,840	2,340	1,420	4,660	499	637	1,510
1929	2,960	3,040	1,530	890	4,310	312	480	934
1930	3,240	3,070	1,560	840	2,500	275	332	639
1931	3,235	3,600	1,880	940	2,740	324	324	632
Mean	4,030	4,220	1,990	1,140	3,290	379	521	1,050

* Corrected for storage in Cedar Lake.

In examining table 2 it will be apparent that the c.v. varies with the characteristics of the drainage area and with the rainfall conditions. Generally speaking, for the streams with head waters in the Cascades, the c.v. is approximately 0.20 or less, while streams in the

more arid areas with drainage areas of comparable size have a larger c.v. This is best illustrated in studying the first eight streams in table 2. It is difficult to generalize concerning the reasons for the particular c.v.'s of individual streams, and this could be done with greater profit if a correction could first be applied for the amount of ground storage in the drainage area. Ground storage, as Hazen uses the term, measures the ability of a given drainage area to sustain stream flow during periods of minimum rainfall. Large sand and gravel deposits would increase this quantity, and the term also includes the effect of variation in rainfall distribution and of snow storage. Hazen recommended that a value less than 0.20 be not used. These streams, therefore, have a much smaller degree of variability than he found for most American streams, since a large number of the streams shown in table 2 have a smaller coefficient, and in cases like the Columbia or Baker Rivers the coefficient is much smaller. The head waters of Baker River are supplied from the glaciers and snows of Mount Baker and the high precipitation of the Cascades, and this explains both its low c.v. and its high run-off of 146.5 inches per year over the entire drainage area of 184 square miles. Its maximum rate of discharge of 200 c.f.s. per square mile is also noteworthy. The annual precipitation at the higher elevations in the Cascades is thus seen to be very high, although unfortunately rainfall records at the higher altitudes in this state are exceedingly meager.

At first glance, it would appear that the c.v. increases with the size of the drainage area, as illustrated by the Columbia and Yakima Rivers. The increase in c.v. of the Columbia, however, probably comes from the inflow of streams in its lower reaches from the semi-arid areas where higher c.v.'s are the rule. The Skagit, on the other hand, exhibits the opposite behavior and has a smaller c.v. at Sedro-Woolley than at Ruby Creek, resulting from the inflow of Baker River with its small c.v. between these points. The variations in c.v. at different points on a given stream probably arise from the inflow from tributaries having differing c.v.'s and from their equalizing effect upon each other.

The c.v. as given in the table is for the stream with such ground storage as is found to exist in its drainage area. If the area were impervious these data would be more directly comparable with Hazen's values than they are at present. The smaller ground storage would give a larger c.v. where both of these factors are combined in a

TABLE 2

Run-off data for northwestern streams

NUMBER	RIVER	LOCATION	C.V.	RUN-OFF	DRAINAGE AREA	MEAN ANNUAL FLOW	DISCHARGE: Mean	DISCHARGE: Maximum	DISCHARGE: Minimum	NUMBER OF YEARS RECORD
				inches per year	*sq. mi.*	*c.f.s.*	*c.f.s. per sq. mi.*	*c.f.s. per sq. mi.*	*c.f.s. per sq. mi.*	
1	Skagit	Reflector Bar	0.198	49.6	1,100	4,030	3.66	52.7	0.60	23
2	Sauk	5 mi. above mouth	0.185	80.0	714	4,220	5.91	122.9*	0.97*	17
3	Baker	3 mi. below Baker Lake	0.139	146.5	184	1,990	10.80	200	1.18	21
4	Middle Fork Snoqualmie	Camp 15 near North Bend	0.183	94.1	163	1,135	6.95	105.7†	0.59†	20
5	South Fork Flat Head (Hungry Horse Res.)	Columbia Falls, Mont.	0.237	27.1	1,640	3,290	2.00	28.2	0.15	18
6	Cedar	At Crib Dam	0.262	66.5	77.56	380	4.91			25
7	Chiwawa		0.257	39.0	181	521	2.88	13.3	0.50	18
8	Priest	Above Priest Lake	0.247	24.9	572	1,050	1.83			18
9	Snoqualmie	Snoqualmie	0.164	89.0	375	2,470	6.57	85.3	0.60	23
10	Cedar	Landsberg	0.205	72.1	135	719	5.32	55.5	1.20	30
11	Nisqually	La Grande	0.200	64.6	287	1,370	4.76	58.2	0.64	11
12	Skagit	Ruby Creek	0.190	45.7	978	3,300	3.37	46.7	0.46	20
	Skagit	Sedro Woolley	0.150	73.9	2,970	16,200	5.45	74.1	0.95	16
13	Columbia	Trail, B.C.	0.116	28.6	34,000	72,000	2.11	9.19	0.28	14
	Columbia	Grand Coulee	0.154	20.1	74,100	110,000	1.48	6.64	0.27	14
	Columbia	The Dalles	0.177	11.8	237,000	206,000	0.87	4.93	0.20	50
14	Spokane	Spokane	0.260	22.3	4,350	7,150	1.64	11.24	0.11	37
	Spokane	Little Falls	0.272	17.1	6,380	8,040	1.26	6.49	0.17	16

15	Wenatchee	Leavenworth	0.190	51.0	591	2,220	3.76	28.4	0.42	18
	Wenatchee	Cashmere and Dryden	0.206	39.0	1,200	3,450	2.87	21.2	0.39	13
16	Yakima	Cle Elum	0.197	53.5	500	1,970	3.94	51.2	0.29	21
	Yakima	Union Gap (Yakima)	0.185	18.9	3,550	4,930	1.39	17.95		23
	Yakima	Kiona	0.237	11.24	5,520	4,580	0.83	11.5	0.19	18
17	Willamette	Albany	0.202	39.3	4,860	14,100	2.90	41.0	0.46	19
18	Bull Run	Bull Run	0.205	109.8	102	828	8.10	168.5	0.60	34
19	Rogue	Tolo	0.236	19.8	2,020	2,950	1.46	23.9	0.40	19

* At Darrington with 293 sq. mi. drainage area.

† For drainage area of 173 sq. mi. This maximum may have been exceeded in 1909 and 1910.

Note: The year Oct. 1–Sept. 30 is used except for Nos. 5, 7 and 8, where the records are for Apr. 1–March 31.

common coefficient as in table 2. An attempt was made to estimate the amount of ground storage in the streams under consideration by a comparison of the writer's data with Hazen's tables. In figure 2

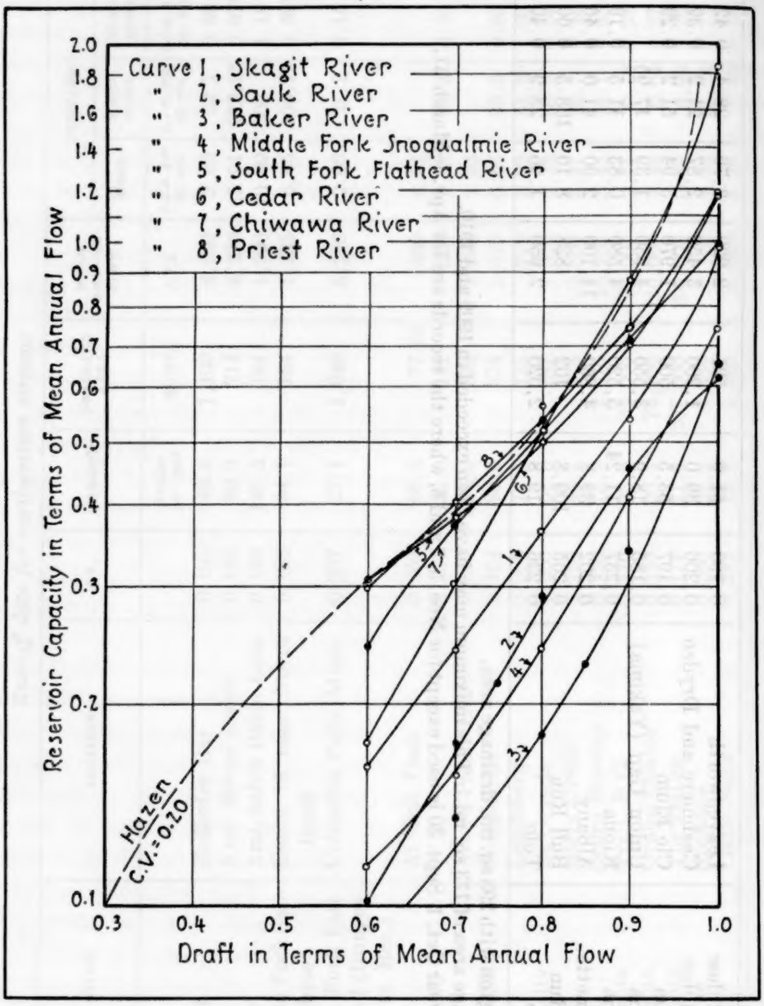

FIG. 1. STORAGE REQUIREMENTS—NORTHWESTERN STREAMS

the data have been assembled in more usable form by constructing curves showing the relation of coefficient of variation to storage requirement for drafts of different proportions of the mean annual

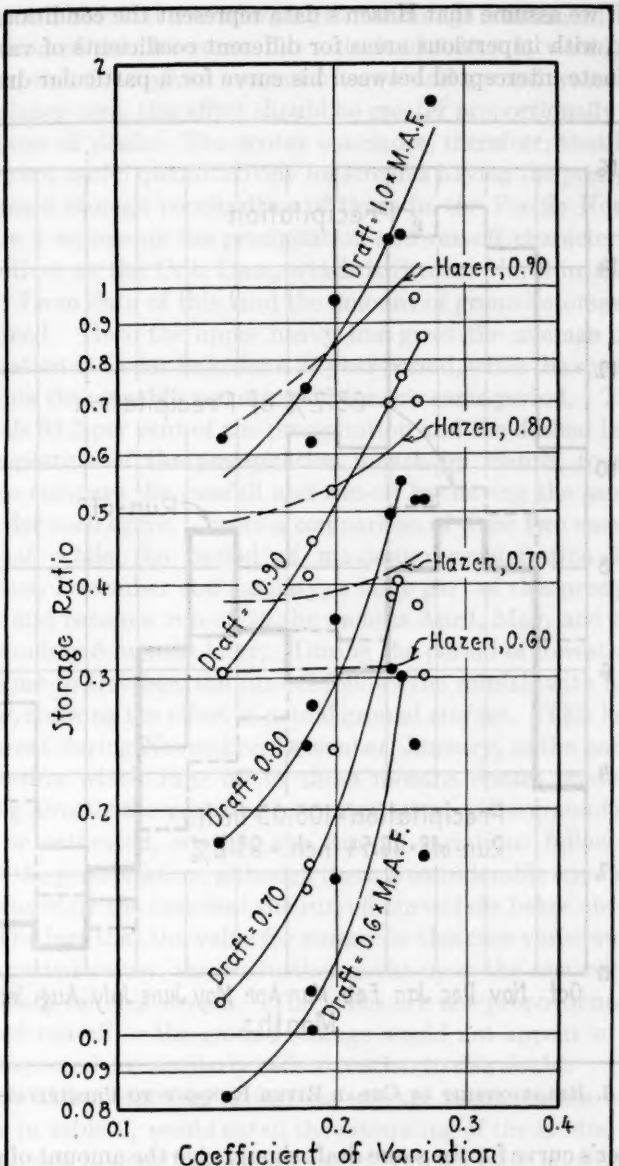

Fig. 2. Relationship of Coefficient of Variation to Storage Ratio

flow. If we assume that Hazen's data represent the conditions to be expected with impervious areas for different coefficients of variation, the ordinate intercepted between his curve for a particular draft and

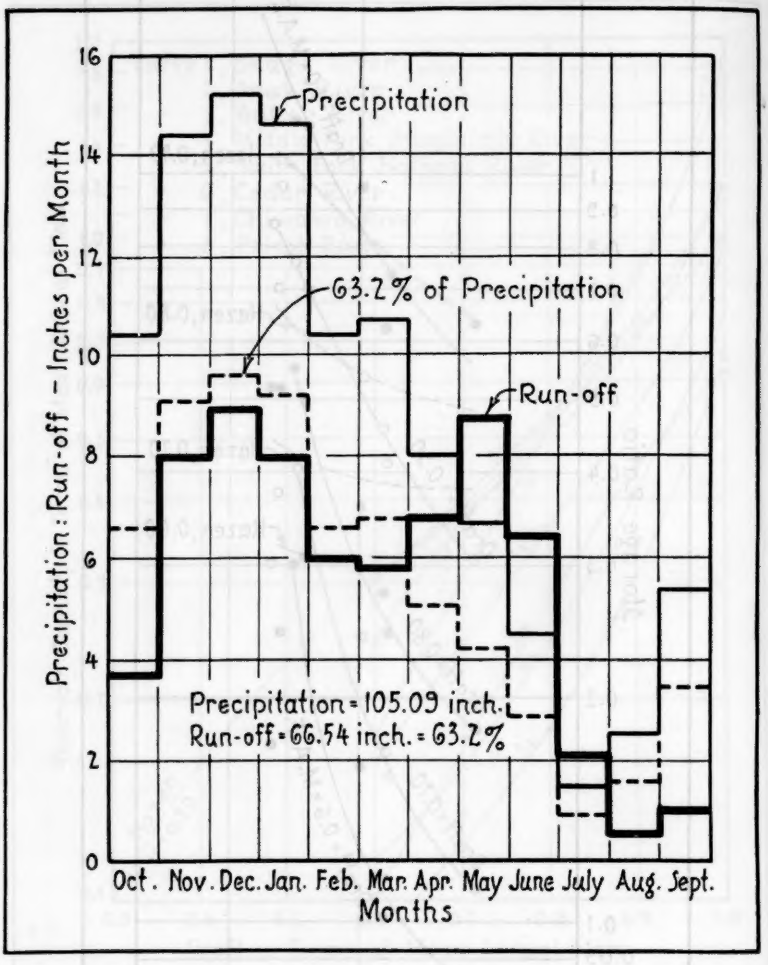

FIG. 3. RELATIONSHIP OF CEDAR RIVER RUN-OFF TO PRECIPITATION

the writer's curve for the same draft should give the amount of ground storage as heretofore defined. This ordinate can be readily translated into number of days storage, following Hazen's method for expressing this factor.. If the two curves for a draft of 0.90 are com-

pared, it will be found that a larger ground storage is indicated than by comparison of the two curves for a draft of 0.80. It would appear, however, that for a given amount of snow and ground water storage in a drainage area, this effect should be greater proportionally for the lower rates of draft. The writer concludes, therefore, that Hazen's data are not useful quantitatively for streams having the precipitation and ground storage peculiarities of those in the Pacific Northwest.

Figure 3 represents the precipitation and run-off characteristics of Cedar River at the Crib Dam, which is Stream No. 6 in the above study. From data of this kind the amount of ground storage can be determined. Here the upper heavy line gives the average monthly precipitation at Cedar Lake for a 25 year period, while the shaded area represents the monthly mean run-off for this same period. This run-off equals 63.2 per cent of the precipitation and the dashed line gives this proportion of the precipitation month by month, to make it easier to compare the rainfall and run-off by having the same total area under each curve. From a comparison of these two curves, it is seen that while the period of maximum precipitation includes November, December and January, a large part of this precipitation is snow and becomes run-off in the months April, May, and June, or approximately 5 months later. During the period of lowest rainfall, say August to October, the run-off follows the rainfall with about 30 days lag, showing the effect of actual ground storage. This lag is not so apparent during November, December, January, as the part of the precipitation which runs off in those months occurs immediately, following directly the variation in precipitation. The ground is either frozen or saturated, so that the run-off variations follow closely those of the precipitation, although there is considerable snow storage, as evidenced by the fact that the run-off curve falls below the dashed line. The fact that the value for storage in this case varies so greatly from expected values, throws further doubt upon the accuracy of the run-off data for this stream. The losses are not proportional to the measured run-off or the ground storage would not appear so erratic. The unexpected c.v. similarly calls attention to this doubt.

To use this method in estimating storage requirements on other streams in table 2, would entail the estimating of the ground storage on the stream in question and the use of Hazen's table above referred to, or direct comparison with the data in figure 2 might be made. For example, assuming that it is desired to determine the storage required for the Spokane River at Spokane. From figure 2, if the

storage were to be determined for 0.9 draft, a c.v. of 0.260 would give a storage ratio of 0.865. A study of the mass diagram for this stream roughly checks this value. On the other hand, the Willamette with a c.v. = 0.202, would require a storage of 0.51 under the same conditions, according to figure 2, whereas the mass diagram shows its storage requirements to be above 0.70. These streams are very different from the majority of the streams included in the study and would not be expected to be readily comparable, as heretofore suggested. If, however, several studies had been made of streams of somewhat similar characteristics, it is probable that sufficiently accurate results could be obtained by determining the c.v. and taking values of storage from curves similar to figure 2, or tables similar to Hazen's, without going through the laborious process of constructing the mass diagram.

In conclusion, the writer wishes to point out that this method shows for the streams in table 2, including a wide variety of characteristics of drainage area and precipitation, that higher maintainable flows can be obtained for a given storage ratio than on streams in other sections of the United States. It is probable also that the economic development of higher percentages of the mean annual flow may be possible for Northwestern streams. It is readily apparent that the dependable flow without storage is greater per square mile and the variability less than for similar streams elsewhere. This applies particularly to the streams with head waters in the Cascades and the mountains of British Columbia. This method therefore, has the advantage of making easier the comparisons between different streams by simplifying the method of using data for adjacent or similar streams. It is believed that as more data are published containing studies of streams throughout the United States it will be easier to determine whether Hazen's curves are adequate as standards for comparison and what factors will be applicable to streams having any particular characteristics. The method does not, however, appear to be as useful in studies of the power possibilities of a stream, since it applies particularly to a uniform draft.

(Presented before the Pacific Northwest Section meeting, May 13, 1932.)

SPECIFICATIONS FOR FILTER CONSTRUCTION

By Wm. E. Stanley

Hydraulic Engineer, Pearse, Greeley and Hansen, Consulting Engineers, Chicago, Ill.

The character of the specifications covering the construction of filters may be determined by various factors such as:

(a) The function of the specification.
(b) The time, energy and talent available for preparation.
(c) The care and amount of detail employed in the preparation.

The efficiency of filter plant operation and many of the items which make for convenience in operation relate directly to the character of the construction specifications, particularly as regards the many mechanical appurtenances which are necessary in a modern filtration plant.

GENERAL TYPES OF SPECIFICATIONS

Specifications for a water filtration plant may be classed generally under two types:

(a) The brief descriptive type indicating the general kind of plant desired, the numbers and functions of various units, the capacity basis of design and the like. Such specifications have been used for receiving proposals for the furnishing of a complete operating plant with construction details to be developed by the builder.

(b) An extended or complete specification covering in minute detail the quality and the quantity of materials to be included, the quality of workmanship, the exact type and character of every item required in the plant. Such specifications accompanied by detailed construction drawings indicate exactly what is to be built and proposals for the construction work are received accordingly.

The first type was used in the early history of water filter construction and is now frequently used in connection with plans for financing the construction of filtration projects.

The second type is in more general use and has served for the construction of most of the modern filters. Such detailed specifications have a great many advantages, some of which are:

(1) The design of the plant is determined definitely for the owner by his engineers in advance of receiving bids.

(2) Closer competition in bidding is obtained.

(3) Contractors can determine closely the character and extent of the work required resulting in close, intelligent and comparable bids.

(4) The construction work can be divided in several contracts according to type of construction work, thereby obtaining better work at lower prices from qualified contractors.

TREND IN SPECIFICATIONS

The trend in the development of filter plant specifications has been toward more specific statements covering in greater detail the various items to be included in the construction. Formerly complete filtration plants were bought ready to operate, later many of the major elements in the plant were specified very briefly. The more recent specifications include greater detail.

FILTER PLANT CAPACITIES

The capacity to be provided in a filter plant and the major elements of the plant are matters relating to the design rather than to the specifications. However, some indication of the capacity items is usually included in the specifications. The capacities of the minor elements, devices and mechanical appurtenances are more definitely a part of the specifications. The boundary line is not very definitely fixed as between those capacity elements which are design and those which may be properly and definitely a part of the specification.

RELATION OF PLANS AND SPECIFICATION

The construction drawings and the written book of specifications are closely related. Often much of the specification is written directly upon the drawings. It is better to keep the amount of explanatory notations required on the plans down and to cover everything as completely as possible in the written specification book. In any event the plan drawings and the written specifications should be consistent and taken together should be complete.

MAJOR CONSTRUCTION ITEMS

The major items which enter into the construction of a filtration plant may be listed as follows:

(a) Excavation, backfill and cleanup
(b) Concrete work

(c) Building construction
(d) Piping, valves and sluice gates
(e) Filter underdrains
(f) Wash water troughs
(g) Filter sand and gravel
(h) Filter control equipment
(i) Chemical feed and sterilization devices
(j) Laboratory arrangement, equipment and supplies

The first three items are not peculiar to filter plant construction and do not particularly affect the efficiency of operation of the plant except from the point of design. First class work should be required and somewhat greater care must be taken with the specifications covering the concrete work to insure watertightness and to prevent deterioration by weathering which may be somewhat more pronounced in certain cases because of the exposure of thin walls with water pressure on the inside.

The items of piping, valves, sluice gates, filter underdrains, washwater troughs, are more or less special to water filter plants and their proper design and construction will have influence on the efficiency of the plant. These items usually may be covered more especially in the design drawings, rather than in the specifications.

The remainder of the construction items including the filtering material, the filter control equipment, the chemical feed and sterilization apparatus and the laboratory control devices, cannot be readily or adequately covered by the details on the construction drawings. They are items which have a very definite influence on the plant operation and require particular attention in the writing of the construction specifications.

FILTER SAND AND GRAVEL

The filter bed is made of two elements, the supporting gravel, and the filtering material or sand. The function of the gravel is solely that of a support for the sand. The function of the sand is to remove as completely as possible those impurities remaining after preliminary treatment, and the sand should have such qualities as to perform form this function most effectively and economically. Actually the service required of the filtering sand is more complex than that of a mere strainer. It should perform as an efficient strainer, frequently under wide variations in quality of applied water. On the other hand, it is desirable that there be as low a resistance to flow as possible and

it is necessary that the filtering material have such characteristics as to permit it to be readily cleaned during backwashing. These requirements are contradictory in effect.

The primary characteristic of the filter gravel is its proper grading as to size from top to bottom. The lower layers should be relatively large so as to offer a minimum of resistance to wash water flow. The several layers should be of such variation in size that the upper layers will not drop into the void spaces of the lower layers and the top size should be small enough to hold up the sand. Recent difficulties with the filters at the new St. Louis plant has emphasized this.

The gravel should also be relatively free of lime, or other easily dissolved materials. Many specifications require an acid test as a measure of the soluble material. It has been difficult in this region to obtain gravel entirely free of acid soluble material.

The question of the proper specification for filter sand is a very live subject just now and has been much discussed during recent months. A Committee of the American Society of Civil Engineers has been giving attention to the subject for the last five years. Recently a committee of the Purification Division of this Association has been appointed.

The standard specification of the past has usually stated in effect that the filter sand shall be composed of hard durable grains, free from clay, loam, dirt and organic matter and shall have an effective size between such and such sizes and a uniformly coefficient of not more than 1.65. The better prepared specifications may have included some limitations as to the amounts of flat sand grains, calcium and amounts of fine material.

The terms effective size and uniformity coefficient have been used for so long that many water works men and material supply companies use them as a matter of course and sometimes with little or no understanding as to the exact meaning of the terms.

It may not be out of place here to point out again that the terms effective size and uniformity coefficient were coined by Hazen in connection with studies of slow sand filters where the sand bed is not disturbed by back washing. Since the principal standards for filter sand, namely, effective size and uniformity coefficient were originally developed in connection with the slow sand filter, possibly some modifications in such standards might be obtained to fit the conditions better which arise in the operation of the rapid sand filter. However, the present yardsticks of effective size and uniformity co-

efficient are so generally accepted that any proposed change must be considered with great caution.

Many engineers, including the late Allen Hazen, specify sand sizes by seive separations and do not use the terms effective size or uniformity coefficient. The Detroit Department of Water Supply recently developed a specification for filter sand including some unusual clauses. The sizes of the sand were set up quite closely including both lower and an upper limit for effective size and uniformity coefficient. Tolerances were included, permitting some fine material at a penalty of reduced price and of the contractor being required to remove the fine material by washing and scraping.

Generally speaking, there is a tendency to use coarser sand than formerly. This has been due in part to the acceptance of chlorination as a safe and practical method of bacterial removal and thus reducing the function of the filter bed to that of producing a clear water.

The filter sand constitutes a relatively small item in the construction cost of a filter plant and a relatively major item in the plant operating efficiency. Thus considerable care is warranted in specifying the desirable characteristics and of including provisions for being rather strict in obtaining the type of sand specified.

SELECTION OF MANUFACTURED EQUIPMENT

Included in the filter equipment are a considerable number of devices of standard manufacture by several companies. Among other things this would include gages, meters, valves and gates, agitators, etc. In few cases is it possible to set up, definitely, the best and most economical equipment for a specific case. Furthermore, this would eliminate competition and eventually tend to increase the cost of the work.

A more desirable method is to require and to specify, with considerable exactness, the functions to be performed by each device including limiting degrees of accuracy and also require that all such equipment be subject to approval before installation and subject to test after erection. In this way the contractor must include satisfactory equipment for the purpose intended, but it is to his interest to obtain the best possible price from several manufacturers.

If it is found, upon further investigation, that certain refinements can be added to the equipment proposed by the contractor which will increase its efficiency, durability or convenience of operation

beyond that specified, then these can be added by additional payment, but at no actual increase in cost for the same equipment as specified. The equipment as proposed must satisfy the conditions imposed before it is acceptable and any increase in cost should be only for items not essential but desirable. This method has been found to operate satisfactorily in practice.

The application of the method involves a very careful study of the functions to be performed by the equipment and a careful and conscientious effort by the engineers in making comparisons of the several makes of equipment the contractor may submit for approval.

It is frequent practice to require a performance guarantee covering the equipment which extends over some period of time following the completion of construction. It is proper that the manufacturers should assume some responsibility for the performance of his equipment. It is not fair, however, to require the contractor to guarantee the efficiency or overall performance of the purification plant.

FILTER CONTROL EQUIPMENT

Control equipment for filter operation consists of three principal parts:

(1) Control of rate of filtration
(2) Control of rate of wash water application
(3) Control of operation of valves

There are several satisfactory devices of standard manufacture available for the control of rate of filtration. All of these vary in detail, but are on the same principle so a general specification as to performance, with proper guarantee, will procure satisfactory equipment. Refinements of detail and convenience can be added as found desirable.

Control of wash water rates is largely a function of design rather than specification. The use of wash water tanks or a wash water pump will control the necessity for other devices. A rate controller of a type similar to the controller used on the filters is a desirable device for use with a tank, whereas this is not so necessary with a pump as the pump discharge rate can be easily controlled by gate valves.

There are three possibilities for the control of filter valves, namely, hand operation, hydraulic operation and electric operation. A choice between these is a matter of design controlled to a large extent by the

size and number of filter units. For very small units up to say 0.5 m.g.d. nominal capacity per unit, it is generally considered that hand operated valves are satisfactory. For larger units manual operation becomes less desirable. Electric control so far has been found in most instances too expensive for general application. Hydraulic valve control involves several details requiring special attention such as pressure available for actuating the valves, protection against sticking or jamming, valve and piping arrangements for controlling the actuating water, operating tables on which controlling levers are mounted, indicating devices for position of the gate, etc. The specifications should be very definite as to these items as this part of the plant is used more frequently than any other and trouble with these devices can be very bothersome to the operator.

CHEMICAL FEED AND STERILIZATION DEVICES

The choice between dry feed and solution feed devices is largely a question of design, related by judgment to the particular case in question. Solution feed devices consist essentially of an orifice box with the necessary piping and valves for control. These devices are mostly special and are subject to detail on the plans. Materials for floats, orifices, pipes, valves, fittings, etc., should be carefully specified to resist chemical action and corrosion.

There are a number of dry feed machines of standard manufacture, all designed to accomplish the same ultimate result by various methods. The arrangement of machines, methods of filling, layout of piping, etc., are subjects for design study and should be tied in with the specifications so that machines proposed will fit the layout. Here again it is desirable to have general specifications for machines requiring certain capacities, accuracy and adjustments and leaving the details of construction for careful consideration and approval.

Sterilization is one of the most important processes in water treatment especially in those locations where the supply contains sewage pollution. The relation between sewage treatment and water treatment is becoming ever closer and must be given careful consideration in the development of all projects.

Early methods of sterilization included the use of hypochlorite as an agent, but this has been generally supplanted by liquid chlorine because it is more convenient and economical. Two types of standard machines are available for applying liquid chlorine, the solution and dry feed types. Each has its advantages and these must be carefully

balanced against cost and performance leading to a choice. More recently the use of ammonia with chlorine has been tried out.

LABORATORY

In general, specifications for laboratory appurtenances should be limited to major equipment such as benches, tables, cases, plumbing, piping connections, hoods and similar items which can be furnished in competition by a number of companies. The arrangement of such equipment is a function of design for space alloted for laboratory rooms and the extent of laboratory control work which is to be carried on. Minor equipment and supplies are items subject to preference by the laboratory technician and, where possible, should be selected by him. If necessary to include all expenditures in contracts, it has been found satisfactory in several cases to instruct the contractor in the specifications to include in his proposal a definite sum of money to be used for the purchase of such articles, the choice to made at some later date. Any difference between actual cost and cost allowed to be credited to the contractor or the owner as the case may be.

SUMMARY

Specification practice for water filtration plants have changed considerably during recent years. The trend has been toward stating more exactly and in some detail the quality of workmanship or function of the device to be required and to leave as little as possible to be decided upon after the award of the contract.

The preparation of construction plans and specifications in considerable detail and carefully tying them together results in closer bids and lower costs as there are less uncertainties as to what is to be required for the contractor to include in the contingency items in his bid.

(Presented before the Wisconsin Section meeting, October 26, 1931.)

THE INFLUENCE OF STANDARDIZATION ON CONSTRUCTION SPECIFICATIONS

By Julian Hinds

(*Chief Designing Engineer, The Metropolitan Water District of Southern California, Los Angeles, Calif.*)

As a part of a general study of standard specifications being sponsored by the American Water Works Association, an attempt is made in this paper to discuss the influence of standardization on specifications for construction work. Inquiry reveals the existence of an impression that "standardization," as it is generally recognized, is not particularly applicable to such specifications. The extent to which this feeling is justified depends somewhat upon what is meant by a standard specification.

It is possible to go to a filing case and take out a ready-made specification for a shipment of reinforcing steel, cement, cast iron pipe, and many similar or more complicated products, but the same procedure cannot be followed for the construction of a dam, tunnel, or similar work. The individual needs of each such job must be studied and allowed for. However, construction work does not differ as much from other work in this respect as is sometimes supposed. Although a whole job cannot be reduced to definite standard procedure, many of the parts of it can be, and usually are.

Essentially, a specification is a description of an object, or piece of work, designed to convey to the prospective producer as concrete an idea as possible of the buyer's wants. The owner needs to know in advance that the finished work will fulfill his requirements, and the builder, who at the time the specification is prepared is not known to the specification writer, before submitting a price must know what is expected of him, so that he may fairly estimate his costs. The task of preparing the description is facilitated if the buyer's wants are limited to commonly known or established things or processes. It is comparatively easy to describe a thing already fully understood by all parties to the description. As the degree of advanced understanding diminishes, it becomes more and more diffi-

cult to prepare the description and the chance of misunderstanding increases.

It is difficult to write a complete description of the simplest object, without drawing heavily upon common knowledge of things and processes. Without exhaustive research, no reader of this paper could write a specification sufficiently clear and complete to permit an intelligent and educated human being having had no previous contact with the world, except through books, to construct, without experimentation, such a simple thing as an ordinary short-handled shovel. But to write such a specification for the use of a shovel manufacturer is an easy matter, because it is possible to presume upon his knowledge of the shovel-making business. In fact, there is no need in this age to write a specification for a shovel, unless some special qualification is desired.

A complete specification for an earth dam would be enormously involved. It would be an encyclopedia of practically every known industrial art, from the intricate details of cement manufacture, mining, metallurgy, etc., to the production details of the salt and yeast used in the bread in the contractor's mess. Its compilation would be impossible and, if compiled, the specification would be useless.

Fortunately such a specification is not required. Dams are not built by people who know nothing about them. Machinery for building the fill, for example, is purchased from people who are in the business of manufacturing such equipment and who know much more about it than can be written into any specification, and who in addition have at their command exhaustive files of standard specifications for various parts of their products. Unless there is some reason for doing so, the specification for the dam need not even mention the excavating machinery, and thus important dependence is placed upon industrial standardization and special standard specifications, although the dam itself remains an individual structure, the component parts of which must be individually described.

The cement that goes into concrete parts of such a structure must be specified, as its qualities affect the finished structure. However, it is not necessary to write a complete specification for this product. Reference is simply made to some standard specification for cement, which in turn is little more than a commercial description of certain qualities which enables a competent manufacturer to give the pur-

chaser what he desires—what he asks for, plus many other things unknown to the purchaser but necessary in a successful cement.

The same procedure applies to all metal and other materials entering into the structure. It is difficult to estimate the number of "standard" specifications that contribute directly or indirectly to a large construction job.

Speaking more directly to the point, the standardization of construction specifications may be assumed to refer to agreement upon the processes by which specific construction results may be secured. For example, many specifications have been written for the compacting of an earth fill, but upon examination great divergence is revealed among them. Nevertheless, as experience is gained, there is a tendency to weed out the undesirable processes with a trend toward a similarity of requirements.

Possibly a standard specification, or a set of standard specifications, for such fills will eventually be evolved. As soon as knowledge on the subject is sufficiently crystallized it is likely that some committee of this Association, of the American Society of Civil Engineers, the Bureau of Standards, or other similar organizations, will be called upon to classify the accumulated material and prepare a series of alternative specifications which may be used as standards for such work.

By slow degrees other similar construction processes are likewise being standardized. The manufacture of concrete, for example, is now in the process of standardization. In fact, standard specifications for concrete have already been proposed for general use, but since they have not yet been fully accepted, they should perhaps be considered as more or less tentative. With the improvements that are being made constantly, and growing familiarity with these improvements, it is hoped that reasonably full acceptance of such standards is imminent. The wide variety of materials available in in different localities for use in concrete makes complete standardization unlikely.

One of the great benefits of standardization is that it promotes a complete understanding without the necessity for involved descriptions. This applies not only to the building up of a store of written standards to which reference may be made, but perhaps more particularly to the accumulation of trade knowledge that goes with unification of processes. If a certain operation is always performed in the same manner, its correct performance will ultimately become a matter

of common knowledge, eliminating uncertainty and confusion in specifications and promoting efficient execution.

It is not sufficient that specification writers agree among themselves to adopt a given standard, but to be effective the resulting specification must be understood and accepted by the construction fraternity. A worker seldom produces the best results when compelled to use a method which he believes to be incorrect, or super-theoretical, even though such method may in fact be superior to the one he prefers. The personal element in execution cannot be ignored. It is difficult to change from one standard of procedure to another, or to create a new standard, suddenly. As a rule, departure from accepted practice must be made gradually, and in carefully proven steps.

It is often difficult to secure satisfactory results from a newly proposed or novel method of construction, even though it may be good, until it has come to be generally understood by construction men and contractors and they have come to recognize its benefits. This is especially true where the new method upsets established routine or is not suited to the equipment in general use. This is illustrated by the limitation of the water content of mass concrete. For many years chuting was the accepted method of placing such concrete, and wet mixes were the rule. Then the value of a reduced water content came to be recognized in the laboratory, but its application in the field was resisted for a time because it upset routine. Finally, new methods of handling are being introduced and there is now a tendency toward drier mixes.

A successful specification writer must possess not only an adequate technical knowledge of the subject to be treated, but he must be conversant with the knowledge possessed by others in regard to it. He must know what items must be written down in detail and to what extent dependence can be placed on standardized knowledge and practices. It is never possible for the writer to know everything about the structure to be built, and if such complete knowledge were available it could not practically all be written down and followed.

It is necessary to depend largely upon the unwritten knowledge of the prospective constructor and his workers. Keen discrimination is particularly required on public works where the necessity for explaining and justifying every action sometimes makes it difficult to select the contractors as carefully and freely as can be done on private work.

To write a specification which can be readily understood by an

honest, experienced, well informed, and competent contractor is quite a different matter from writing one which cannot possibly be misunderstood by an inexperienced or dishonest one. Specifications for public works must, to a certain extent, serve the latter purpose, and for that reason are likely to seem unreasonably complicated to those who have had to do only with private works.

The foregoing statements are too elementary to be of value to an experienced specification writer and too indefinite and incomplete to be of much assistance to a beginner. The writer hesitates to attempt to set down any definite rules of procedure, partly because of a feeling of incompetence, and partly because of the danger that such rules would be subject to possible misuse.

One rule that can be given with safety is that no important construction specification should be finally issued until it has received the approval of a competent engineer experienced in the preparation and execution of similar specifications. Competent legal advice should likewise be secured. Such expert advice will usually follow the preparation of a draft, by members of the owner's staff, or some one employed specifically for such purpose.

The first step in the preparation of such a draft is the determination of the nature and extent of the work to be done. An attempt should be made to collect all possible physical data likely to affect the nature or cost of the work. This material should be studied before beginning the specification. The results of such studies should be recorded in memoranda and reports and made available for use during the preparation of the plans. The work to be done should be thoroughly understood and the necessary drawings prepared before the specification is begun. A specification is a description, and a description cannot be properly written by one not fully conversant with the thing to be described.

Following the assembly of physical data, a collection of available standard specification and of clauses from special specifications for similar work should be made. Such a collection will serve as a valuable guide when the actual writing of the specification is begun.

AVAILABLE STANDARD SPECIFICATIONS

If any materials are to be described or covered by the proposed specification, it is quite likely that a search of available standard specifications will reveal many standard forms suitable for the pur-

pose at hand. Such standards have been prepared for many kinds of merchandise and construction material.

The various departments and bureaus of the U. S. Government have issued standard specifications for such a wide variety of articles as "beef liver," potatoes, hay, many kinds of paints, axe handles, various kinds of clothing, practically all kinds of metals, small tools, construction equipment, cement, glue, and so on, almost without limit. Lists of such of these as have been printed can be secured from the Superintendent of Documents, Government Printing Office, Washington, D. C. Lists of unpublished standards can be secured from the Army, Navy, Bureau of Standards, Forestry Service, Bureau of Reclamation, and so on. Many of these standards are intended for the use of commercial departments of the Government and are of little interest to the construction engineer. However, many of them are of value.

Many technical societies likewise issue standard specifications. The American Society for Testing Materials, in particular, specializes in this work, and maintains standards for all of the principal construction materials. These specifications have come to be recognized as authoritative and may be used merely by reference, without being actually copied into the construction specifications. Reinforcing steel, structural steel, and cement specifications are notable examples of these standards.

The American Water Works Association, the American Society of Mechanical Engineers, the American Institute of Electrical Engineers, and other societies and private organizations are engaged in the production of standard specifications for equipment and materials in which their members are particularly interested. Copies of these standards can be obtained through public and private libraries, or from the societies themselves.

Wherever practicable, it is preferable to use accepted standards bodily, by reference, rather than attempt to copy extracts into the construction specification. This is not always feasible, due to variable conditions.

In addition to these recognized standard specifications, nearly every large organization having to do with construction works has some kind of a semi-standard set of specifications covering all of its ordinary operations. A collection of such semi-standards, from as many sources as possible, and a comprehensive file of specifications for important construction jobs, is an indispensable part of every

specification-writer's library. The judicious study of such a collection affords ideas, lessens the liability of omitting some essential matter, and assists the writer in expressing his ideas in accepted form. The continued selection of the best clauses from such specifications tends toward ultimate standardization, although the process may at times seem slow.

STEPS IN PREPARATION OF SPECIFICATION

After all the required data have been assembled and a definite decision has been reached as to the nature and extent of the work to be undertaken, the preparation of the written specification should begin. Perhaps the first task undertaken should be a simple physical description of the various parts of the proposed works. The first draft of this description may be largely spontaneous, but ultimately it should be compared in detail with the previously collected clauses and standard specifications and made to conform to them as nearly as practicable. However, standard clauses should not be copied blindly. Each clause used for comparison should be carefully studied. Any unusual provision encountered should be scrutinized with care before being used, to disclose any possible special reasons underlying its origin.

Sufficient inquiry into the past history of many seemingly desirable clauses will show that they have caused trouble in actual application, or that they were born of some previous difficulty and designed to fit a particular set of conditions. Not only must these difficulties be known to the future users, but the underlying circumstances must be fully understood in order that the defects may be eliminated without danger of introducing other more objectionable ones. The relation of the proposed project to projects covered by the specifications used as guides should be studied and new clauses devised to meet any new requirements.

Except in the case of a new organization or the introduction of a new line of work into an old one, specifications rarely spring up suddenly. They are more likely to be the result of a gradual growth from some modest past beginning. Sections and clauses are added from time to time to meet new conditions or to eliminate newly discovered difficulties. Unfortunately, old clauses are not always dropped as rapidly as they become obsolete, the result being, in many cases, a more complex structure than is necessary.

The reluctance to delete supposedly obsolete clauses is not alto-

gether unreasonable. In fact, all changes from an accepted procedure should be made with care. A properly written specification is a very delicately balanced document. A change made to remove a known simple difficulty may unexpectedly introduce others of a more serious nature. Only thorough study, supported by careful review by one or more specialists in specification writing, can insure a safe final result. Each questionable clause should be scrutinized from every possible point of view.

The borrowing of clauses is no doubt the safest way to proceed in the writing of a specification, but it is essential that the task be undertaken with care and understanding. It is not safe for a novice to prepare a specification by the simple process of compiling apparently suitable clauses from previous examples.

Generally, all conclusions drawn from studies of data should be excluded from the specifications. Actual facts may be presented where desirable or necessary, but usually the bidder should be required to make his own deductions from records of observed facts. The opinions of expert advisers on geological, physical, and legal matters must be at times be furnished to the bidder, but this is best accomplished by making available to him special signed reports, outside of the specifications. Any direct expression of opinion by the owner or his agent in the specification itself may cause embarrassment, especially where the specification later becomes part of a contract, as it usually does.

Specify results

Insofar as practicable, it is preferable to specify results, rather than means of producing results. Contractors are likely to resent being told in too great detail how a given task is to be performed. The specification writer cannot know in advance all of the facilities and the personal idiosyncrasies of the prospective bidder, and regardless of competency and experience, the writer and the constructor are not likely to have identical ideas as to the best means of accomplishing the desired result. In fact, it is often impossible to determine the best detail procedure until after the identity of the constructor and his facilities and experience are known.

One of the advantages of letting work by contract is to permit the owner to avail himself of the contractor's knowledge. This advantage should not be sacrificed by allowing the contractor too little freedom of action. Needless details, if incorrect, may lead to high

bids, contractor's loss, and trouble for everyone. Even if sound, they may cause high bids, or the elimination of desirable competition. On the other hand, too few details may lead to unsatisfactory results. If the writer fully understands the work to be done and is confident that certain details of procedure are necessary or desirable, he may require that they be followed. Also, details are sometimes required to guard against fraud in case the work is unknowlingly let to an unscrupulous contractor. The specification should not unduly restrict the honest contractor, on the one hand, and must not allow too much latitude to a dishonest one, on the other.

The rigid enforcement of unusual or extreme restrictions is difficult. The engineer must of necessity be given the final say in the interpretation of the specifications, and constant pressure is brought to bear to have any apparently unreasonable clauses ignored. Too rigid enforcement of provisions which, although entirely within the legal discretion of the purchasing agency, appear unreasonably extreme, tends to destroy the coöperation of the contractor, which is an important element in the successful completion of a job.

The inspector's task is made easier if the contractor is left as free as possible in all non-essential matters, so that the necessity for coercion and compromise is reduced to a minimum.

Unfortunately the idea of specifying results rather than methods has not yet been fully accepted in many lines. For example, the important factors in concrete for most purposes are its strength, durability, density, and volume change characteristics. But no specification is ever written setting forth only these factors. Many manufacturing details are always specified. This results from a lack of assurance that meeting the known physical requirements will always give a satisfactory product, and the difficulty of knowing at a sufficiently early date that these qualities are being met by the concrete placed in the structure.

In the case of an earth enbankment, as a further example, no standard method of determining the desirable degree of compaction, after the work is completed, has yet been generally accepted. It is therefore usual to specify the method of compacting in considerable detail, to give the contractor as good an idea as possible of what will be expected of him. It is to be hoped that eventually this, and many other operations, can be specified in such manner that the contractor will be free to proceed in any reasonable way so long as the desired final result is obtained.

As an additional example, it is not always possible to control definitely the percentage of overbreak in rock excavation. With equal care in excavation the overbreak will vary according to the nature of the materials to be excavated. The overbreak also varies with the care exercised in excavation. Where the cost of removing overbreak material is borne in full or in part by the owner, it is necessary to have some definite check upon the contractor's operations. Such a check usually takes the form of specific directions in regard to blasting or other construction operations. Such specific directions are excusable only to the extent that they are necessary. Where certain excavation procedures are known to be injurious, or where the owner is confident that certain procedures are desirable, and worth any possible additional cost, it is proper to give specific directions.

It should be borne in mind, however, that any needless limitation may increase the contractor's bid. The contractor may differ entirely with the specification writer as to the most advantageous procedure, and the specification writer may be wrong. Regardless of whether he is wrong, imposing his ideas needlessly on the bidder may result in a higher quotation. Unnecessary details about which the writer is not fully informed should be particularly avoided. The determination of the exact extent, in any given case, to which restrictive clauses should be carried, requires careful discrimination.

OTHER PHASES

From a strictly technical point of view, the physical description of the work to be done and the necessary standard references to materials and processes complete the specification. However, every construction specification ultimately forms part of a contract. It is therefore usual to include in the specification many clauses which are in fact of a contractual nature, but which are necessary for the determination of the bidder's costs.

The bidder must know when and how he is to be paid for the work, whether a bond will be required of him, how rapidly the work must be pushed forward, etc. If there are any special requirements as to the employment of labor, working Sundays, provisions of safety devices, design of camps, etc., he must know about them. If there are hazards connected with the work he must know to what extent the risks are to be borne by him, so that he may add the value of such risks to his price.

It is not theoretically necessary to call attention to all laws that are to be complied with, the assumption being that the bidder is familiar with such laws. However, it is advisable to call attention to any special or unusual legal requirement, particularly if out of state bids are to be solicited. This should be done in such a way as to leave it clear that laws not mentioned are likewise to be observed.

Particular attention should be called to any possible financial limitations, especially where funds are to be derived from future appropriations from legislative bodies, or from taxes not yet levied.

When all of these things and others which suggest themselves are provided for, the specification becomes a rather formidable document. Every attempt should be made to have each section complete, incapable of being misunderstood, and as simple and concise as possible. The number of sections included should be ample, but useless provisions should be eliminated. The language should be simple and clear. It is important that the writer understand the technical capacity of the prospective performer of the work so that the specification may be written in language that will be readily understood. The use of highly technical or involved legal terminology should be avoided.

(Presented before the California Section meeting, October 26, 1932.)

INDICES OF THE SANITARY QUALITY OF SWIMMING POOL WATERS

By M. P. Horwood, B. S. Gould and H. Shwachman

(Department of Biology and Public Health, Massachusetts Institute of Technology, Cambridge, Mass.)

The widespread increase in the number of swimming pools in the United States, and the favor with which swimming is regarded as a refreshing form of exercise, indicate that the swimming pool is here to stay. Riker (1) reports that in 1900 there were only 67 public swimming pools in the United States. In 1913, however, this number had increased to 540, and at the end of 1929, the number exceeded 5,000. In New Jersey alone, at the end of 1929, there were approximately 75 indoor pools and 60 outdoor swimming pools. From the public health point of view, the primary question involved is, how can these pools be operated and supervised so that all danger to health may be eliminated?

In the United States the problem has been attacked from three points of view.

1. How should a swimming pool be designed and constructed, in order to afford the maximum safety, convenience and comfort, commensurate with the cost?

2. How should the water be treated in order to insure its purity all the time? This has led to the widespread use of the recirculating system, in which the pool water is used over and over again after treatment by coagulation and filtration, and after disinfection with liquid chlorine or hypochlorite of lime.

3. The chemical and bacteriological supervision over the water in the pool in order to determine the residual chlorine, the turbidity, and the final bacterial content.

In addition, special regulations have been adopted concerning the use of the pool, which have aided in maintaining it in a suitable and sanitary condition. For example, all bathers are required to take a shower and to empty the bladder before entering the pool. Bathers suffering from skin infections or respiratory diseases are prohibited

from using the pool. Nude bathing is recommended. Spectators are required to use the special areas reserved for them; and finally, bathers are instructed in the proper use of the pool in order to minimize the danger of infection.

It is unnecessary here to discuss the essential requirements for the satisfactory construction and operation of swimming pools. Such standards were set up in 1926 by the Joint Committee on Bathing Places of the American Public Health Association and the Conference of State Sanitary Engineers, and are available to any student who is interested (2). The important question to be considered here is to what extent are the available bacterial standards, indices of the suitability of swimming pool waters for bathing purposes? Also, which standard, if any, should be employed in the sanitary supervision of swimming pools?

That drinking water may be a vehicle of disease has unfortunately been demonstrated only too often. Since the diseases that are transmitted through drinking water are usually intestinal in character, it was only natural that the standards for the bacterial purity of drinking water should be based on the total count and the B. coli content. That is the accepted practice today, with perhaps more emphasis on the presence or absence of B. coli than on the total count.

That swimming pool waters may be responsible for the transmission of disease has long been suspected, but has not been subject to such ready proof as is available where drinking waters are involved. Furthermore, the significance of intestinal disease is greatly diminished as far as its potential transmission through swimming pools is concerned. Although bathers may take water into their mouths, and may actually swallow an occasional mouthful, the amount so consumed is unquestionably infinitesimal. Furthermore, the water of a swimming pool is rarely, if ever, contaminated by the fecal discharges of the bathers, and while urinary contamination may occur more frequently, it probably does not represent a serious source of contamination. Therefore, if the water in the pool is safe when first introduced, there is little likelihood that it will become infected with pathogenic intestinal bacteria during the course of its use. This view is supported by Grierson (3) who has recently reviewed some of the existing literature concerning the transmission of disease through swimming pools and polluted bathing waters. In this review he says that "intestinal infections (obtained through bathing) appear to have occurred through bathing in polluted rivers, or in swimming pools

which derive their water from such rivers, and the cause of the disease has been due to general sewage pollution of the water rather than infection from bather to bather."

On the other hand, the opportunities for oral and respiratory infections seem to be very much greater, for the water of the swimming pool enters the mouth, nose, ears and eyes, and thus may prove to be a vehicle of disease through these portals of entry into the body. Diseases that have been especially associated with the use of swimming pools in recent years, are rhinitis, tonsilitis, conjunctivitis, sinus infections, the common cold and other respiratory diseases.

In the review noted above, Grierson (4) enumerates certain specific epidemics of intestinal disease which is it believed were due to infections with B. typhosus, B. dysenteriae, or B. proteus obtained through bathing in infected water. Reference is also made to specific epidemics of conjunctivitis, otitis media, sinusitis and various other sinus and respiratory diseases, as well as to venereal diseases and skin infections. Grierson actually determined the longevity of specific pathogens in sterile and non-sterile tap water, and found that when an agar slant culture is transferred to 300 cc. of sterile tap water, specific pathogens survived, as shown in table 1.

TABLE 1

ORGANISM	PERIOD OF SURVIVAL
Staphylococci	At least 144 hours (6 days)
Hemolytic streptococci (scarlatinae)	At least 144 hours (6 days)
Meningococci	2 hours but not 2.5 hours
Gonococci	1. 5 hours but not 2.0 hours
Pneumococci	At least 144 hours
B. diphtheriae	72 hours but not 96 hours
B. typhosus	At least 144 hours

However, when similar concentrations of the same organisms were prepared with non-sterile water, they could be isolated after 48 hours, only with the greatest difficulty.

Since the possibility of contracting sinus infections, eye and ear infections, skin infections and respiratory disease from the use of swimming pools seems to be materially greater than the possibility of contracting intestinal disease, it would appear that the method of determining the bacterial quality of the water in a swimming pool, should be different from that which is employed to determine the

sanitary quality of a drinking water supply. In testing drinking water supplies bacteriologically, great emphasis is logically placed on the presence or absence of B. coli. It is questionable, however, whether the same index should be employed in evaluating the sanitary quality of swimming pool waters.

Mallman (5) found that B. coli were capable of multiplying in swimming pool waters during the night, when the pool was not in use, whereas streptococci did not increase, and concluded that the latter were more reliable indices even of intestinal pollution. Furthermore, there seemed to be a direct relationship between the concentration of streptococci in the pool and the existing pollution. In 1930, Mallman and Gelpi (6) described a method of examining swimming pool waters for streptococci, by planting the water in lactose broth, incubating at 37°C. for 48 hours and at 20°C. for 72 hours, and then examining smears made from the sediment for coccus forms, including streptococci, after decanting the supernatant.

Cary (7) reports that Mrs. C. O. Wiltsie, former senior bacteriologist for the Detroit Health Department, found that after examining 175 smears made from the sediment in lactose broth tubes that had been inoculated with water from 36 pools, 40 percent showed cocci and only 5 percent showed B. coli. The examinations were also correlated with the free chlorine content of the water. When the free chlorine was 0.1 p.p.m. or less, 59 percent of 116 smears examined showed the presence of cocci. When the free chlorine content was 0.2 p.p.m. or more, 5 percent of the 59 smears examined showed cocci, and none showed B. coli. When there was no free chlorine present, cocci were found in 73 percent of the smears, (number not given), and B. coli in 13 percent. It would seem, therefore, that streptococci served as a more satisfactory index of pollution in swimming pool waters than B. coli, especially where bacteria from the mouth, nose and skin are concerned.

In the light of the above information, it is of interest to inquire into the bacterial standards for swimming pools, which are now in use. The standard suggested by the A. P. H. A. Committee on Swimming Pools in 1926, stated that "not more than 10 percent of the samples covering any considerable period shall contain more than 100 bacteria per cc. when cultivated in nutrient agar or litmus lactose agar at 37°C. for 24 hours, and that no single sample shall contain more than 200 bacteria per cc." The 20°C. count is optional, and the recommendation says that at 20°C. after 48 hours, not more than

10 percent of the samples covering any considerable period shall contain more than 1,000 bacteria per cc., and no single sample shall contain more than 5,000 bacteria per cc. The standard for B. coli says that not more than 2 out of 5 samples collected on the same day, or not more than 3 out of any 10 consecutive samples collected on different dates shall show a positive presumptive test in 10 cc. of the water when the pool is in use. No reference is made to the use of streptococci or other coccus forms as a more suitable index of pollution of swimming pool waters, than B. coli. However, it is recommended that "the amount of available or excess chlorine in the water at all times when the pool is in use, shall not be less than 0.2 p.p.m. nor more than 0.5 p.p.m."

In New York City samples of water from swimming pools that come under the supervision of the Board of Education are collected twice each week, and sent to the Prospect Laboratories in Brooklyn where the drinking water of New York is analyzed. The minimum bacterial standards employed in judging the sanitary quality of the swimming pool waters is as follows (8):

The bacterial count. Not more than 10 percent of the samples covering any three months period shall contain more than 500 bacteria per cc. when incubated for 24 hours at 37°C. on nutrient agar, or litmus lactose agar.

Test for the B. coli group. Not more than 2 out of 5 one cc. samples collected on the same day for special purposes, or not more than 3 out of 10 consecutive 1 cc. samples of water collected periodically when the pool is in use, shall show a positive test indicating the presence of the B. coli type of organism.

The bacterial standard employed by the New Jersey State Department of Health is identical with the A. P. H. A. standard noted above (9). In order to determine the value of the A. P. H. A. standards as an index of the sanitary quality of swimming pool waters, all of the analyses made by the Boston Health Department from 1920–1931 on local swimming pools were examined. The Boston Health Department conducts a weekly bacteriological analysis of all swimming pools in Boston and it is estimated that there are between 10 and 20 in operation all the time, depending on the season of the year.

The results of this examination show that there is no correlation between the total count and the B. coli incidence. Some of the reports show high total counts with a low B. coli incidence, while others show low total counts and a high incidence of B. coli. Furthermore,

after a careful analysis of the laboratory examination on swimming pool waters made in Boston in 1930, it is obvious that the majority of the pools would be condemned on the basis of the proposed A. P. H. A. standards for total counts, in spite of the fact that on inspection most of them were found to be functioning in a satisfactory manner. It raises the question, therefore, whether the standards suggested by the A. P. H. A. committee are not after all too severe, and hence unnecessarily stringent for purposes of health protection.

In Detroit each swimming pool is given a "sanitary rating" based on the results of the bacterial analysis of the swimming pool water. The "sanitary rating" is obtained by determining in any series of samples the percentage of total bacterial counts in excess of 200 per cc. and the percentage of 10 cc. portions positive for B. coli. The

TABLE 2

Bacterial results on swimming pool waters, Detroit, 1925–1930

YEAR	NUMBER OF POOLS	TOTAL NUMBER OF SAMPLES EXAMINED	PERCENT OF SAMPLES WITH ZERO BACTERIAL COUNT	PERCENT OF SAMPLES WITH BACTERIAL COUNTS OF 200 OR LESS	PERCENT OF SAMPLES WITH NO B. COLI ORGANISMS
1925	30	4,555	27.0	72.6	82.9
1926	37	3,829	39.6	79.4	90.5
1927	39	3,752	57.3	84.1	93.9
1928	44	4,246	63.5	91.3	97.0
1929	53	5,061	65.2	93.1	96.7
1930	55	6,144	72.0	96.4	98.6

two percentages are averaged arithmetically, and multiplied by 10. The result is subtracted from 1,000, and the new result is called the "sanitary rating." If the rating falls below 900, the pool is classified as unsatisfactory. Such pools are subject then to constructive criticism and are not necessarily condemned. A pool that is considered "unsafe," however, is closed immediately until conditions are corrected (10).

It will be observed that the determination of the "sanitary rating" is based on the bacterial standards for swimming pools suggested by the A. P. H. A. Committee. While no reference is made to the presence or absence of streptococci, or other coccus forms, excellent results have been obtained in improving the sanitary condition of the swimming pool waters in Detroit. This is confirmed by the data presented in table 2 (11).

Since the sanitation of swimming pools is one of the subjects discussed in Municipal Sanitation at the Massachusetts Institute of Technology, and the students are required to make an original investigation, two of the students (B. S. G. and H. S.) elected to survey some of the more prominent swimming pools in Cambridge and Boston, and to correlate the results of the sanitary survey with tests for residual chlorine, total counts, B. coli content, and streptococci, particularly of the hemolytic variety. Ten pools were carefully examined in this way, the work being done in March, 1931.

Samples were collected in sterile 250 cc. glass-stoppered bottles in accord with the directions given in Standard Methods of Water Analysis, and examined within four hours after sampling. All samples were collected during the latter part of the week. Each sample was examined for total count on nutrient agar at 37°C. after 24 hours; for lactose fermenting organisms at 37°C. after 24 and 48 hours in 1 cc. and 5 cc. amounts, and for total counts and hemolytic bacteria on blood agar at 37°C. after 24 and 48 hours. Positive tests for lactose fermenters were run through the partially confirmed test, using eosin methylene blue agar plates, and typical looking B. coli growths were examined as smears and then inoculated into lactose broth again. The presence of gas in the second set of lactose broth tubes, together with the appearance of typical B. coli-like organisms in the microscopic examinations, were considered to be confirmatory evidence of the presence of B. coli.

In addition, smears were made from the hemolytic colonies appearing on blood agar and examined microscopically for coccus forms, especially streptococci. The technique of Mallman and Gelpi for the isolation of streptococci, consisting of the examination of smears made from the sediment in lactose broth tubes, was also used.

The blood agar plates were made from defibrinated horse blood, using 6 cc. per 100 cc. of sterile, melted nutrient agar, cooled to 40°C. Each sample of swimming pool water was examined quantitatively for residual chlorine by the ortho-tolidin test.

Two series of samples were collected, the first being examined for total count and B. coli, the second for total count, B. coli and streptococci. The results of the first series of tests are recorded in table 3.

Several interesting items can be observed from table 3. The first is that with one exception, the total counts were relatively low; second, the samples were relatively free of B. coli; and third, in the

one case where B. coli was isolated, it could not be correlated with the total count.

TABLE 3

Total count and B. coli incidence in swimming pools of Cambridge and Boston, March, 1931

SWIMMING POOL	TOTAL COUNT ON NUTRIENT AGAR AT 37°C. AFTER 24 HOURS	PRESENCE OF B. COLI IN VARYING AMOUNTS OF WATER	
		1 cc.	5 cc.
Harvard University	0	0	0
Radcliffe College	150	0	0
Cambridge Y. M. C. A	300	0	0
Boston Athletic Association	300	0	0
Roxbury Boys' Club	200	0	+
Cabot Street	2,000	0	0
University Club	40	0	0
Boston Y. M. C. A.	200	0	0
Boston Y. W. C. A.	50	0	0
Winsor School	40	0	0

TABLE 4

Total counts and incidence of hemolytic streptococci and B. coli in swimming pool waters obtained in Cambridge and Boston, March, 1931

SWIMMING POOL	TOTAL COUNT ON NUTRIENT AGAR AT 37°C. AFTER 24 HOURS	HEMOLYTIC STREPTOCOCCI PRESENT IN 1 CC. AMOUNTS	B. COLI PRESENT IN 1 CC. AMOUNTS
Harvard University	2	0	0
Radcliffe College	100	+	0
Cambridge Y. M. C. A.	350	+	0
Boston Athletic Association	250	+	0
Roxbury Boys' Club	400	+	0
Cabot Street	1,500	+	0
University Club	50	0	0
Boston Y. M. C. A.	150	+	0
Boston Y. W. C. A.	50	0	0
Winsor School	40	0	0

A second series of samples was therefore collected and examined for total count, hemolytic streptococci and B. coli, in order to ascertain whether there was any correlation between these indices of pollution. The results are given in table 4.

The interesting items brought out in table 4 are, first, that there is

no correlation between the presence of B. coli in 1 cc. amounts of swimming pool water and hemolytic streptococci in the same quantity of water; second, that there is no correlation between the B. coli content per cc. of water and the total count; and third, that there is a significant correlation between the presence of hemolytic streptococci per cc. of swimming pool water and the total count.

While the above tests were being made, simultaneous analyses were performed on the same samples of water in order to isolate streptococci by the Mallman and Gelpi method. Tests were also made to determine the amount of residual chlorine. The correlation

TABLE 5

Correlation between the Mallman and Gelpi test for streptococci, the presence of hemolytic streptococci, the total count and residual chlorine in swimming pool waters found in Cambridge and Boston, March, 1931

SWIMMING POOL	PRESENCE OF STREPTOCOCCI AS DETERMINED BY MALLMAN AND GELPI METHOD	PRESENCE OF HEMOLYTIC STREPTOCOCCI	TOTAL COUNT ON NUTRIENT AGAR AT 37°C. AFTER 24 HOURS	RESIDUAL CHLORINE
				p.p.m.
Harvard University	0	0	2	0.5
Radcliffe College	+	+	100	0.2
Cambridge Y. M. C. A	++	+	350	0.25
Boston Athletic Association	++	+	250	0.2
Roxbury Boys' Club	++++	+	400	0.2
Cabot Street	+++	+	1,500	0.0
University Club	+	0	50	0.3
Boston Y. M. C. A	++	+	150	0.25
Boston Y. W. C. A	0	0	50	0.3
Winsor School	0	0	40	0.25

of these two items with the total counts and the presence of hemolytic streptococci, recorded in table 4, are given in table 5.

The data in table 5 indicate that a high degree of corrrelation exists between the Mallman and Gelpi test for streptococci and the presence of hemolytic streptococci isolated from blood agar plates. It is also apparent that there is a high degree of correlation between the presence of streptococci and the total count, whereas previous evidence presented in this paper indicates no similar relation with B. coli in 1 cc. amounts of water. Finally, it should be mentioned that those swimming pool waters which had from 0.3 to 0.5 p.p.m. of residual chlorine, showed very low bacterial counts, no hemolytic

streptococci, and only very few streptococci as determined by the Mallman and Gelpi test.

In every instance, where a sample of swimming pool water was collected, a sanitary survey of the pool was likewise made. Specific attention was directed at such items as construction, method of water purification, sanitary supervision of bathers, and general administration. All of the pools recirculated their water and employed coagulation, filtration and disinfection. Harvard University employed liquid chlorine for disinfection, but the other pools used either bleaching powder or sodium hypochlorite. Chlorination was not employed in one pool—the Cabot Street Pool—but this was a temporary condition only, and a fresh supply of disinfectant was being awaited daily. However, because of this deficiency, the counts were very high and hemolytic streptococci could be isolated very readily.

TABLE 6

Results of the sanitary survey of certain swimming pools in Cambridge and Boston, made in March, 1931, correlated with the total count and residual chlorine

SWIMMING POOL	TOTAL COUNT ON NUTRIENT AGAR AT 37°C. AFTER 24 HOURS	RESIDUAL Cl_2	ADMINISTRATION	AGE OF POOL	SANITARY MAINTENANCE	SANITARY CONTROL OF BATHERS
		p.p.m.				
Harvard University	2	0.5	Excellent	Modern	Excellent	Excellent
Radcliffe College	100	0.2	Excellent	Old	Excellent	Excellent
Cambridge Y. M. C. A	350	0.25	Fair	Old	Fair	Poor
Boston Athletic Association	250	0.2	Good	Old	Good	Excellent
Roxbury Boys' Club	400	0.2	Fair	Modern	Poor	Poor
Cabot Street	1,500	0.0	Fair	Old	Poor	Poor
University Club	50	0.3	Excellent	Modern	Excellent	Excellent
Boston Y. M. C. A.	150	0.25	Good	Modern	Good	Fair
Boston Y. W. C. A.	50	0.3	Excellent	Modern	Excellent	Excellent
Winsor School	40	0.25	Excellent	Modern	Excellent	Excellent

Table 6 indicates that where the administration, sanitary main-

tenance and sanitary control of bathers in the operation of swimming pools are excellent, the bacterial results are especially satisfactory. In such cases, however, the residual chlorine was usually inclined to be high.

The results of this general inquiry into the sanitation of swimming pools seem to point to the following conclusions:

1. Since intestinal diseases are spread through the use of swimming pools only very rarely, if at all, some index of the sanitary quality of the water other than B. coli, should be employed.

2. The total count on nutrient agar at 37°C. after 24 hours, represents the best and most simple index of the sanitary quality of the water in the pool.

3. The blood agar plate test for hemolytic streptococci or the Mallman and Gelpi test for streptococci in the sediment of lactose broth should be included in the routine control of the sanitary quality of swimming pool waters, together with the total count on nutrient agar at 37°C. after 24 hours, and the test for B. coli should be omitted from the official requirements in the supervision of swimming pools.

4. The standard for total count in swimming pool waters should be modified to read "not more than 10 percent of the consecutive samples examined within a period of three months should show more than 200 bacteria per cc. on nutrient agar at 37°C. after 24 hours. However, if in the opinion of the health officer, the count at any time is inordinately high, and the sanitary survey indicates a potential menace to health, the pool should be shut down at once, until it is again brought into satisfactory operating condition."

5. The absence of B. coli in one and five cc. amounts of swimming pool water is no indication of the absence of serious pollution.

6. There is no correlation between the presence or absence of B. coli in one and five cc. amounts of swimming pool waters, and the total count, the presence of hemolytic streptococci or the presence of streptococci in general.

7. The residual chlorine in swimming pools should preferably be maintained between 0.3 and 0.5 p.p.m.

8. The supervision over swimming pools should include a periodic—perhaps monthly—sanitary survey, of the type outlined in this paper.

BIBLIOGRAPHY

(1) Riker, I. R.: Outdoor Bathing Places, New Jersey Public Health News, Jan.-Feb., 1930.

(2) Report of the A. P. H. A. Committee on Swimming Pools, covering standards of design, construction, equipment and operation. Amer. Jour. of Public Health, v. 16, Dec., 1926.

(3) Grierson, A. M. M.: Observations on the hygienic condition of public swimming baths. Jour. of Hygiene, v. 30, No. 1, April, 1930.

(4) Ibid.: No. 3.

(5) Mallman, W. L.: Amer. Journ. Public Health v. 18, p. 771, 1928.

(6) Mallman and Gelpi: Bull. 27, Mich. Engineering Experiment Station, 1930.

(7) Cary, W. H., Jr.: Swimming Pool Review, Detroit Health Dept., Aug., 1929.

(8) Anonymous: The Sanitation of Swimming Pools, Weekly Bulletin, N. Y. C. Dept. of Health, v. 20, No. 40, Oct. 10, 1931.

(9) Ibid.: No. 1.

(10) Anonymous: Detroit's Swimming Pools During 1930. Weekly Review, Detroit Health Dept., March 7, 1931.

(11) Ibid.: No. 9.

CLEANING OF DISTRIBUTION MAINS

By Orla Casad

(*Superintendent, Water Works, Merced, Calif.*)

The cleaning of distribution mains has been common practice for considerable time in the east and Middle West with very advantageous results. However, there has been but little of this work done out here. It is the purpose of the writer to tell you of his own experience along the same lines in the City of Merced, California, with the hope that it will be of value to any one experiencing any of the following troubles with the distribution of water:

1. The lack of carrying capacity in the mains for domestic supply, irrigation and fire protection.
2. The presence of dirt and discoloration in the water.
3. Obnoxious odors and objectionable organisms.

The following results were obtained from the cleaning of a 16-inch cast iron gravity main, 31,620 feet long, having a static head of 81 feet which had been in service 27 years.

Before cleaning, the carrying capacity of this pipe, 24,000 feet from the intake was 1,220 gallons per minute. After cleaning, the flow was 2,317 gallons per minute, or an increase of ninety per cent. This work was done in 1915. This is a brief description of the type of machine and how the work was carried on.

A water propelled machine was used. After cutting the pipe, the machine was inserted, the pipe permanently made up and the water turned on. A certain amount of water was allowed to pass the machine in order to carry ahead of it the dirt and incrustation which had been cut from the pipe and ground up by the machine.

The water, machine and cleanings were carried to the surface of the ground through a riser pipe. The water was then shut off and the riser pipe removed and the pipe at that point reconnected. This operation was repeated until the work was completed.

In 1921 we cleaned 41,729 feet of 6-inch cast iron pipe. Practically all of this pipe had been in service for 33 years and more than half of its carrying capacity had been destroyed by incrustation. A sample

of the pipe cleaned that had been in service only 18 years showed more than one half of its carrying capacity had been lost. A different type of machine was used for the cleaning. Instead of a pressure type, a cable machine was used, as pressure machines are not used in small pipe.

In cleaning pipe with a cable machine, it is necessary to pass a $\frac{3}{16}$-inch cable through the pipe by means of a special carrier. This is done by cutting the pipe, inserting the carrier and reconnecting the pipe, then turning on the water to force the carrier through the pipe for a distance of 1,000 to 1,200 feet. This seems to be about the proper distance for cleaning of a pipe at one shot. I cleaned one section 2,000 feet long without any trouble, although the practice is not advisable. After the small cable has been put through the pipe a $\frac{3}{8}$-inch cable is connected to the small cable and pulled through the pipe by means of a windlass, and the cleaning machine is connected to the $\frac{3}{8}$-inch cable and pulled through in the same manner. The water machine and cleanings are carried to the surface of the ground through a riser pipe in the same manner in which the pressure machine was handled. The results of this work were very satisfactory as the carrying capacity of the pipe was restored to that of new. Five men were employed 21 days in the cleaning of the 41,729 feet of 6-inch pipe and very few interruptions to the service were made.

With proper supervision and equipment, pipe that has lost any part of its carrying capacity can be restored to that of new pipe without injuring the coating. It would no doubt be better practice to install pipe specially treated to prevent corrosion if the character of the water is of a corrosive nature. Our experience has proved to us that the cleaning of distribution mains pays, and old mains practically can be kept performing the work of new pipe at small cost.

No general figures of cost can be given, as it is necessary to take into consideration, in making up an estimate of cost of water main cleaning, the location of the work, transportation charges for men and equipment, whether work is located in open country or in congested districts, the amount of the work, the layout of mains to be cleaned, with the location of valves and time required for complete shut-off and whether pipe must be cleaned in short stretches or long, due either to layout or conditions. The depth of pipe and nature of digging also influence the cost.

(Presented before the California Section meeting, October 28, 1931.)

HOW METERS ARE DAMAGED

By C. W. Winkle

(*Superintendent of Distribution and Transportation, Indianapolis Water Company, Indianapolis, Ind.*)

During the year 1931, 2381 meters, which were qualified as broken were removed from service. This number represents 3.5 per cent of all meters in service that year. Of these, 189 were frozen, 1044 were burnt, and the remainder of 1148 meters were removed merely as "broken." This latter group includes meters with broken register glass or dial hands broken off, meters stuck with gravel in them, meters stuck because of worn parts, and meters removed from the test bench when it was found that they did not register properly. During the year 1930, 1125 frozen meters were removed.

Although nearly as many meters are damaged by freezing as by any other one cause, we need not go into detail on this particular phase. It will suffice to say that meters are frozen because of inadequate protection, or through negligence on the part of the resident. We are making every effort to install meters in protected places, and if the basement or cellar of any house does not afford suitable protection, the meter is set in a pit in the yard. The total number of frozen meters can be, and is being reduced by setting meters only in acceptable locations, and by warning customers to give the meters in their basements adequate protection.

A burnt meter is the result of hot water from a hot water heater backing up into the meter. When the pressure in the heating boiler becomes greater than the water pressure in the service line, hot water is forced back into the meter. In such a case, the disc piston becomes warped, and the meter does not register properly, or fails to function altogether. It sometimes happens that the hot water backing up into the meter warps the disc piston to such an extent that the meter continues to run, but does not register properly. To all outward appearances, the meter is operating satisfactorily. It is only when this meter is removed and tested that the damage is ascertained. On a full stream test, the meter with a slightly warped disc will register

about 100 percent. On the smaller flows, however, the percentage decreases rapidly until a point is reached on low flows where the meter fails to register. The average test of such a meter is about 75 percent. It can readily be seen that with such meters in service, we are not collecting the amount really due us, and it is to the interest of every man in the Company that those who do come in contact with the meters (usually meter readers and service men) be on the lookout for slow-running meters.

Frozen and burnt meters constitute nearly 52 percent of all meters under the so-called "damaged" classification. Among the minor causes for broken meters is the intrusion of sand, gravel, cinders, or some like substance, into the meter. During the years previous, we have had a few main breaks in which quantities of foreign matter have gotten into the mains. While these mains were cleaned as thoroughly as possible before they were repaired, even after a number of years, gravel and cinders appear in meters which can be traced to some previous break in the main. Foreign matter lodging in the meter causes the disc piston in the measuring chamber to stop suddenly, and the pressure shock, transmitted to the working parts, breaks one or of the sensitive parts.

Meters sometimes develop leaks at the stuffing boxes, and if allowed to continue, the water will corrode the mechanism of the clock, and, in a great many cases, the meter fails to register because of a broken clock register, caused by the corrosion from a leaking stuffing box.

Sometimes it happens that yarn, which is used in caulking main line joints, gets into the meter and stops its operation. Again, solder or particles of lead get into the service line through carelessness on the part of the plumber who is instilling a new water service, or making a repair to an old service line. Matter of any sort has practically the same effect on the working parts of the meter, and like any other machine, the meter fails to run when broken. Water continues to pass through, but the amount does not register on the clock.

Service lines into buildings are often installed larger in size than the demand at that time might indicate. This is done with the expectation of an increase in demand in a few years. The minimum rate depends upon the size of the meter installed. Of course, the consumer does not want a meter larger than he actually needs, nor does he want one which cannot supply his needs at the peak demand

period. If the demand is small, and the meter, in proportion, large, the consumer is paying an excessive minimum rate. It is to our interest to furnish him all the water he cares to use. At the same time, there is a possibility that small flows do not register accurately on large meters, which means that we should be very careful that we do not set oversize meters on our customers' lines. We are making every effort to coöperate with the officials of the factory, or whatever the building happens to be, to determine just what the maximum demand will be, so that a meter might be installed that will furnish a sufficient supply of water with the highest possible accuracy of registration.

In cases of this sort, when the demand increases beyond the delivering capacity of the meter, breakage often results. The measuring piston is moved at a higher rate of speed than the rated capacity of the meter allows, and it is inevitable that something give way. This situation is unavoidable at times, because of the difficulty in gauging the demand of the particular building.

Where there is a $1\frac{1}{2}$-inch or larger meter installed, the volume and pressure are sufficiently strong to break some part of the meter when a valve or faucet is closed very suddenly.

We experience very few cases of broken meters from the above causes in the torrent type meter. This type meter is only set on services where a very large quantity of water must be drawn in a very short time, as a railroad standpipe. These meters are being replaced with compound meters, or are being compounded so that small flows can be registered accurately.

In conclusion, we are making every effort to reduce the number of broken meters to the minimum. Through newspapers and printed circulation, as well as through the service men and the meter readers, consumers are giving proper precautions to avoid frozen and burnt meter recurrences. Again, with more experience and improved methods of repairing service and main line leaks, and the laying of new mains, we are reducing the amount of foreign matter that gets into the mains and service lines, and later into the meters. Recent improvements in types of service pipes and fittings also prevent particles in the service lines. Details are important, and the subject of broken meters is one detail to which we must give proper consideration.

(Presented before the Indiana Section meeting, March 10, 1932.)

ABSTRACTS OF WATER WORKS LITERATURE[1]

FRANK HANNAN

Key: American Journal of Public Health, 12: 1, 16, January, 1922. The figure 12 refers to the volume, 1 to the number of the issue, and 16 to the page of the Journal.

The Indole Titer of Gersbach. E. L. KRUGERS DAGNEAUX. Chem. Weekblad, 28: 66–7, 1931. From Chem. Abst., 25: 1869, April 20, 1931. Fact that colon bacilli cease to give indole reaction with lapse of time is no disadvantage, as when observed it shows that pollution occurred long before and that there is no longer danger of typhoid contagion.—*R. E. Thompson.*

Reaction Between Metal Receptacles and Foodstuffs. I. Relation of Metals to Fluids: Corrosion. B. BLEYER AND J. SCHWAIBOLD. Biochem. Z., 230: 136–45, 1931. From Chem. Abst., 25: 1913, April 20, 1931. Corrosion of various metals by water, etc., has been studied by authors. Extensive findings are recorded in tables.—*R. E. Thompson.*

The Hardershof Water Purification Plant at Königsberg, Germany. G. SATTLER and R. BRUCHE. Gas-u. Wasserfach, 74: 73–6, 101–4, 128–32, 1931. From Chem. Abst., 25: 1926, April 20, 1931. Details of plant constructed in 1929–30 and of its operation are given, with numerous illustrations. The yellow-tinted ground water supply is softened with lime, clarified and decolorized by addition of from 30 to 60 p.p.m. of alum and settlement, and filtered through rapid quartz-sand filter. The older slow sand filtration plant is still in use.—*R. E. Thompson.*

Experience with the Lime-Baryta Process. A. FREDERKING. Die Wärme, 53: 943–9, 1930. From Chem. Abst., 25: 1927, April 20, 1931. Results of number of investigations on water purification installations are presented. Effect of addition of lime and baryta and limit of softening were determined.—*R. E. Thompson.*

Oligodynamic Dilutions. JULIUS MEYER. Chem.-Ztg., 55: 85–6, 1931. From Chem. Abst., 25: 1865, April 20, 1931. Term oligodynamic, as used in biology, is a misnomer. It was used originally to designate germicidal tele-

[1] Vacancies on the abstracting staff occur from time to time. Members desirous of coöperating in this work are earnestly requested to communicate with the chief abstractor, Frank Hannan, 285 Willow Avenue, Toronto, 8, Ontario, Canada.

kinesis of metals such as was observed when silver or copper plates were placed in aquaria or flower vases. Term, as at present used, includes such germicidal action as is manifested by extremely high dilutions of metallic salts, despite fact that phenomena of telekinesis is entirely absent. Dilutions experimented with are often extremely high. For example, K. König (cf. C. A., 22: 4177) worked with dilutions of silver nitrate and lead nitrate varying from 10^{-2} to 10^{-29}. Latter dilution would contain in the case of lead nitrate 1 molecule in 55,000 liters of solution and in the case of silver nitrate 1 molecule in 28,000 liters. Results thus obtained are scientifically valueless, as such solutions cannot be distinguished from water of highest purity.—*R. E. Thompson.*

Sterilization of Water by Metals. F. Diénert and P. Étrillard. Compt. rend., 192: 185–7, 1931; cf. Compt. rend., 136: 707, 1903; and Lakhovsky, 188: 1069, 1929. From Chem. Abst., 25: 1927, April 20, 1931. Previous work has shown efficiency of silver, zinc, and magnesium in destroying micro-organisms in water. Experiments here reported show that water is freed from *B. coli*, pathogenic organisms, and liquefying bacteria by passing it over a preparation called silver sand, or by agitating it with this material. The organisms disappear more rapidly from water at 37° than at lower temperatures. Presence of sodium chloride retards process of bacterial destruction. It is shown that water thus treated contains less than 0.003 p.p.m. silver.—*R. E. Thompson.*

Purification of Sea Water for Boilers. I. Shik and G. Torosyan. Azerbaldzhanskoe Neftyanoe Khozyaĭstvo, 1930: 10, 104–8. From Chem. Abst., 25: 1927, April 20, 1931. Prevention of calcium sulfate formation is of primary importance. Ninety-five percent of calcium is precipitated by treating 100 cc. of water with 0.317 gram sodium carbonate. Fair results are obtained even with 0.075 percent sodium carbonate.—*R. E. Thompson.*

Boiler Water Conditioning. A Pittsburgh Development. H. N. Welsh and H. A. Jackson. Proc. Eng. Soc. West. Penn., 46: 327–42, 1930; cf. C. A., 21: 3407. From Chem. Abst., 25: 1927, April 20, 1931. Success in boiler operation at high pressures depends upon timely and correct analysis of constantly changing boiler water. Fundamental requirements of boiler water conditioning establish that sufficient amount of dissolved material (carbonate or phosphate) must be maintained to give chemical equilibria; all organic, or saponifiable, material must be eliminated; and minimum concentration of solids must be maintained in feed water. Numerous specific examples, from Pittsburgh area, of such conditioning are cited.—*R. E. Thompson.*

Determining the Quality of Boiler Feed Water by the Electrical Method. Yu M. Kostrikin. Izvestiya Teplotekh. Inst. (Trans. Thermo-Tech. Inst., Russia), 1930: 2, 27–32. From Chem. Abst., 25: 1927, April 20, 1931. Performance of Leeds-Northrup instrument is described.—*R. E. Thompson.*

The Study of the Fouling of Steam Turbine Paddles. Jules Ernould. Chaleur et Ind., 11: 178–80, 1930. From Chem. Abst., 25: 1927, April 20, 1931. Two 4500-kilowatt turbines running on steam at from 14 to 15 kilograms at 320° suddenly gave trouble, after some years of satisfactory preformance, by blades becoming so fouled that rotor was quite out of balance. Two analyses of deposits are given, major constituents being carbonate, calcium, iron oxide, and silica. It was concluded that trouble was due to boiler feed waters. It was finally decided to maintain alkalinity of feed water at 40° to 45°, i.e., about 2 grams per liter. Formation of deposits is discussed and description included of corrosion observed in steel tubes of water reheaters.—*R. E. Thompson.*

An Interesting Water Purification Change. Otto Regel and Albert Wicklein. Chem. App., 17: 207–8, 1930. From Chem. Abst., 25: 1927, April 20, 1931. Summary of changes made in soda-lime softening process and results obtained, together with costs.—*R. E. Thompson.*

A Routine Method for Direct Determination of B. Coli in Large Quantities of Water on a Solid Medium. F. Diénert and P. Étrillard. Ann. services Tech. hyg. Ville de Paris, 1930: From Chem. Abst., 25: 1930, April 20, 1931. In France the commonest test for *B. coli* is by use of 0.1 percent phenol broth. Medium here described is type of eosin methylene blue agar containing 1 liter distilled water, 10 grams peptone (Difco), 2 grams dipotassium phosphate, 10 grams lactose, and 15 grams agar, reaction being adjusted to pH 7.5. Before use, following are added to 25 cc. agar: eosin yellow, 2 percent, 0.5 cc.; methylene blue, 0.5 percent, 0.5 cc.; phenol, 5 percent, 0.25 cc. Highly contaminated waters are plated directly in small amounts. In waters less densely polluted the bacteria must be concentrated. *B. coli* colonies are round, 3 to 4 millimeters in diameter. By transmitted light center of colony is dark violet-blue, this color covering at least three-quarters of area. Periphery is bright gray-blue, sometimes metallic in appearance. By reflected light center of colony appears slightly elevated. Colonies beneath surface are in form of small, dark-blue lentils. *B. aërogenes* colonies are 4 to 6 millimeters in diameter, with deep-brown centers when viewed by transmitted light. By reflected light they do not show metallic sheen. In case of doubt, colonies are picked and inoculated into peptone broth.—*R. E. Thompson.*

Use of Chloramine in Treatment of Pool Water. J. F. T. Berliner. Beach and Pool Mag., 5: 9–10, 1931. From Chem. Abst., 25: 1931, April 20, 1931. Chloramine, unlike chlorine, is not destroyed by organic matter, or actinic rays, nor removed by aëration. Less chlorine is required, reduction as high as 80 percent having been accomplished. Complaints of tastes, odors, and irritating effects, as well as algae and slime growths are reduced. Ammonia is added before the chlorine. Before applying *o*-tolidin test, nitrites, if present, must be destroyed by oxidation with hydrogen peroxide.—*R. E. Thompson.*

Dechlorinating Water. Josef Muchka. Austrian patent, 120, 119, June 15, 1930. From Chem. Abst., 25: 1931, April 20, 1931. In removing excess chlorine from chlorinated water by passing water through active carbon, or

the like, dechlorinating mass is kept sterile by continuously exposing fresh surface to chlorinated water. Water is supplied through injector to top of container for active carbon, and water supply pipe is connected to bottom of container in such way that carbon is sucked up by injector and carried by water to top of carbon mass.—*R. E. Thompson.*

The Measurement of the Adsorption Capacity of Medical Charcoals. HEDWIG LANGECKER. Klin. Wochschr., **9**: 2298–3000, 1930. From Chem. Abst., **25**: 1946, April 20, 1931. Animal charcoals are, in general, better adsorbents than plant charcoals. Fairly good correlation was obtained in comparing adsorbing power of large series of charcoals using methylene blue, morphine, and mercuric chloride as test adsorbates, particularly between last two. Occasional discrepancies occur, e.g., strong adsorption of methylene blue and weak adsorption of other two by same charcoal.—*R. E. Thompson.*

Suitability of Fuels for Diesel Engines. LOUIS R. FORD. Power, **73**: 361–3, 1931. From Chem. Abst., **25**: 1969, April 20, 1931. Since factors which control behaviour of fuel in engine cylinder are not well understood, performance tests must be relied upon in selecting suitable fuels.—*R. E. Thompson.*

Complete Unit Prices for Hoover Dam. Eng. News-Rec., **106**, 505–6, March 19, 1931. Three bids were received by United States Bureau of Reclamation for construction of Hoover dam, power plant, and diversion tunnels on Colorado River at Black Canyon dam site. Bids varied from $48,890,995 to $58,653,107. Contract was awarded on March 11 to lowest bidder. Work must be commenced within 30 days of award. Complete unit prices from 3 bids are given.—*R. E. Thompson.*

A New Method of Phenol Recovery from Gas Liquors. C. SCHÖNBURG. Brennstoff-Chem., **12**: 69–71, 1931; cf. Glückauf, **64**: 436, 1928. From Chem. Abst., **25**: 1970, April 20, 1931. Disadvantages of dephenolizing with benzene are discussed. Tricresyl phosphate has been found to absorb from 10 to 20 times more phenols and also has advantages of non-volatility, insolubility in, and greater specific gravity than, the liquors. Partition coefficients for both are shown diagrammatically for phenol and cresol solutions and gas liquor. Phenols, after absorption, are driven off from high-boiling point tricresyl phosphate by heat, or steam distillation under vacuum. Practice shows that only 4 or 5 parts of tricresyl phosphate are needed to dephenolize 100 parts of liquor containing 3 grams phenol per liter and that a 6 to 8 percent phenol concentration so obtained in phosphate requires only 5 to 6 parts steam per part of phenol for complete recovery. Use for several months shows selectivity for phenols and non-solution of tar. Apparatus, illustrated schematically, is described.—*R. E. Thompson.*

Potable and Industrial Water 1924–1929. Advances, Processes, Literature. H. BACH. Chem.-Ztg., **55**: 17, Fortschrittsber. 1: 1–29, 1931. From Chem. Abst., **25**: 1926, April 20, 1931.—*R. E. Thompson.*

Coefficients of Large Venturi Meters. ARTHUR L. COLLINS. Eng. News-Rec., **106**: 494, March 19, 1931. Criticism of article by S. F. COUGHLAN, writer pointing out that data furnished indicate that error lies principally in salt velocity method used for calibration and not in Venturi tubes. COLLINS emphasizes that in hydraulic measurements, where current meter, Venturi, orifice plate, weir, and pitot tube are used, results are only approximate. Error is often made in referring to "degree of accuracy," when "consistency" is meant. Any one of above methods will frequently show consistency of 1 percent, but there is no assurance of error limit of 1 percent, except under very unusual conditions. It would add greatly to value of work of this character to introduce second method as a check measurement.—*R. E. Thompson.*

Colorimetric Micro Determinations. I. Determination of Bismuth, Aluminum and Zinc. M. TEITELBAUM. Z. anal. Chem., **82**: 366–74, 1930. From Chem. Abst., **25**: 472, February 10, 1931. Determination of aluminum or zinc with hydroxyquinoline (oxine). Treat slightly acid solution (pH about 3) with from 0.3 to 0.5 cc. saturated sodium acetate solution and with 3 or 4 drops of 0.5 percent oxine acetate solution. Heat 15 minutes on water bath, allow to cool, dilute to about 5 cc., and centrifuge for 10 minutes at 2000 to 2500 revolutions per minute. Decant clear liquid, stir up precipitate with 5 cc. water and centrifuge again, repeating process, if necessary, with 2 cc. water. Dissolve precipitate in 1 cc. 4 normal hydrochloric acid and rinse into small flask with 15 to 20 cc. water. Add 0.5 to 1 cc. FOLIN's reagent (4 grams phosphomolybdic acid and 20 grams sodium tungstate heated with 10 cc. 85 percent phosphoric acid and 150 cc. water for 2 hours with reflux condensation; cool, dilute to 200 cc., and filter if necessary) and about 6 cc. cold saturated sodium carbonate solutions. Dilute to 30 cc. and after 25 minutes compare color with that similarly produced with known amounts of aluminum, or zinc. Excellent results were obtained with from 0.0018 to 0.32 milligram aluminum and with from 0.0081 to 0.083 milligram zinc. For zinc determination, use methyl orange as indicator and add oxine acetate solution after heating to 70°. —*R. E. Thompson.*

Army Engineers Oppose Construction of Barrier Against Salt Water in the Sacramento River. Eng. News-Rec., **106**: 440, March 12, 1931. Army engineers have recently completed and transmitted to Congress partial report on Sacramento, San Joaquin, and Kern rivers in California. Survey was made principally to determine advisability of federal participation in construction of headwater reservoirs and in building salt water barrier across upper end of bay from which combined streams flow into San Francisco Bay. Diversions for irrigation have so reduced low flow that salinity of water in delta region has been greatly increased. Salt water barrier is not considered justified from economic standpoint. Structure would cost $50,000,000 and would be no more effective than headwater reservoir on Sacramento River. Of various reservoirs suggested, only one recommended as being of sufficient importance to nagivation and flood control to justify federal aid is the Kennett reservoir. Exclusive of power development, estimated cost of this project is $67,000,000. Reservoir would have storage capacity of 2,940,000 acre-feet and it is believed

that it could be operated to provide dependable flow of 6,000 second-feet in Sacramento River. This would prevent serious incursion of salt water.—*R. E. Thompson.*

Problems in Concrete Dam Design. D. C. Henny. Eng. News-Rec., 106: 431–5, March 12, 1931. Discussion of problem of concrete dam design, with particular reference to uplift. Danger of uplift is now thoroughly realized and various means have been developed for reducing its effect. Chronologically arranged, these consist of: (1) roughening of rock base and concrete surfaces to increase adhesion and resistance to water creep and to sliding; (2) grouting the foundation: (3) introducing drains in foundation and mass concrete; (4) inclining or stepping construction joints upward in downstream direction; (5) employing water stops in construction joints near water face. Grouting along upstream toe does not guarantee against uplift under base, complete closure being rarely affected and usually only diminution of foundation uplift. Vertical drain holes are now common features of design, but requirement that they be straight and accessible for periodical cleaning is too often neglected. Practice is constantly trending toward reduced spacing of contraction joints, 40 to 50-foot spacing being common practice. Cracks have been observed in recent dams where joint distance was only 30 feet. One, and possibly the principal, cause for more extensive cracking may be tendency of cement manufacturers to increase fineness of grinding, which increases heating in mass concrete. Heat control is now receiving attention. Problem is not merely one of avoiding cracks, experiments having indicated that concrete cured under conditions of higher temperatures does not attain strength of test cylinders cured under laboratory conditions. Telemeter measurements in dams have given ample evidence of thermal stresses far greater than any pure loading stress. Problem of heat stresses and cracking in dams can be solved with certainty by using precast concrete blocks. Advantages of latter type of construction are outlined.—*R. E. Thompson.*

Ariel Dam—An Example of Modern Dam Construction Practice. Eng. News-Rec., 106: 435–8, March 12, 1931. Illustrated description of construction of Ariel Dam of Inland Power and Light Company on Lewis River in Washington, a 313-foot structure of thin-arch type with one gravity abutment, 1300 feet long on crest. Foundation was explored by 26,000 feet of drill hole and excavated 125 feet through gravel to bedrock. Stream regulation (for power purposes) provided by 220,000-acre-foot reservoir will be supplemented later by additional storage upstream. Structure contains about 300,000 cubic yards of concrete, including 95,000 cubic yards in gravity section.—*R. E. Thompson.*

Stewart Mountain Dam Completed. Eng. News-Rec., 106: 480, March 19, 1931. Completion of Stewart Mountain dam finishes third stage of power and storage development program on Salt River, east of Phoenix, Arizona, by Salt River Valley Water Users' Association. Features of structure were recently described by C. C. Cragin (New Reclamation Era, 1930, 214). The dam, across canyon 1100 feet wide at crest elevation, forms reservoir of 70,000 acre-

feet capacity. Structure consists of central arched section, 212 feet high above lowest foundation, thrusting against reinforced concrete abutments. Gaps between these abutments and upper canyon walls are closed by gravity sections. Original plan for multiple arch at this site was changed when exploratory work revealed bedrock at depth of 90 feet below streambed.—*R. E. Thompson.*

Early Cellular Corewall Designs. E. H. BURROUGHS. Eng. News-Rec., 106: 622, April 9, 1931. Writer points out that NEWTON L. HALL was originator of the hollow, or cellular, corewall for earth and rockfill dams, and was granted patents for same in 1909 and 1911. Advantages include use of wall with downstream member higher than upstream wall to protect embankment from wave action by draining off such water; substantial reduction in upstream prism; maintenance of absolutely dry prism for downstream section; and prevention of overtopping by floods during construction.—*R. E. Thompson.*

Sloping Cutoff Wall is Economic Feature of Rolled Earth Dam. Eng. News-Rec., 106: 522, March 26, 1931. Full-height sloping concrete cutoff wall, built in rolled earth dam to reduce cost of formwork required by vertical wall, is unusual feature of structure recently completed on Stanford University campus, Palo Alto, California, to increase water supply for irrigation. Construction of fill followed usual practice, particular care being taken in spreading and rolling the material in 4-inch layers. Maximum height of dam is about 70 feet, and 110,000 cubic yards of fill were required. Wall was placed inside fill to eliminate effect of temperature changes. In addition to eliminating formwork required for vertical wall it was considered possible to secure better rolling close to sloping wall without endangering it. Economic advantages of sloping wall, including thinner section, are reported to have more than offset expense of greater wall area. Below fill, the wall extends vertically 6 to 8 feet into bedrock.—*R. E. Thompson.*

Construction Features of Osage Hydro-Electric Development. Eng. News-Rec., 106: 523–8, March 26, 1931. Detailed description of construction of Osage River 129,000-kilowatt hydro-electric project of Union Electric Light and Power Company of St. Louis. Development consists of masonry dam and power station integral with, and forming part of, dam, having combined length of 2543 feet and maximum height above bedrock of 148 feet. Dam proper consists of non-overflow sections totaling 1512 feet in length and spillway section 520 feet long. Self-supporting cellular type cofferdam was employed, designed to withstand unbalanced water pressure of 60-foot head of water on one side.—*R. E. Thompson.*

Jadwin Plan for Mississippi Flood Control Approved by Chief of Engineers. Eng. News-Rec., 106: 448–50, March 12, 1931. No radical change in engineering plan for flood control on Lower Mississippi River is advised by Major General LYTLE BROWN in his report on the river and restudy of project. Certain features of floodway design, such as narrower cleared channels and permanent weirs, are recommended for consideration. Channel rectification on portions

of river is considered worthy of further study and research. Reservoirs are dismissed as too expensive.—*R. E. Thompson.*

Legal Effect of Sanitary Regulations. LEO T. PARKER. Municipal Sanitation, 3: 3, 118, March, 1932. State Law Regulates Pollution of Water. If state law requires permits to be obtained to pollute water, acts resulting in such pollution will be enjoined, if shown that provisions of statute are disobeyed. In THOMPSON *vs.* Craft Cheese Company (291 Pac. 204.), suit was laid to enjoin Cheese Company from pollution of water. Water from floors, machinery, and utensils and whey received by cesspool overflowed into stream. State law provides that it shall be unlawful to discharge, or drain, any sewage, or substance offensive, or dangerous, to health, into any water used for human, or animal, consumption without permit from State Board of Health. Cheese Company had no permit. Court granted an injunction against further pollution. Suits Against Pollution of Water. Example: Sun Oil Company *vs.* ROBICHEAUX (23 S. W. (2nd) 713). Suit was filed against Sun Oil Company and other oil producers for damage caused by pollution of water. Court refused a judgment jointly against the oil companies unless they acted concurrently to cause the pollution. **Jurisdiction of Police Officers.** If state law gives police jurisdiction outside corporation limits, police may enforce all sanitary regulations within area specified. In COURSEY *vs.* City of Andalusia (134 So. 671.), facts were that state law authorized police jurisdiction within one and one-half miles of corporation limits. This statute was contested, but was later held valid. **Legal Nuisances.** LUTZ *vs.* Department of Health of Commonwealth (156 Atl. 235). Owner of piggery, where swine are kept and fed on garbage, was served notice to abate and remove a certain nuisance arising from insanitary conditions. Evidence showed that piggery was situated upon marshy land subject to overflow, resulting in water pollution. Court ruled that piggery was subject to abatement as public nuisance.—*R. E. Noble.*

The Use of Activated Carbon for the Removal of Tastes and Odors at Saginaw, Michigan. ALFRED ECKERT. Reprint of article presented at 6th Annual Michigan Conference on Water Purification at Petoskey, Mich., October 15, 1931. Use of powdered activated carbon at Saginaw Filtration Plant gave excellent results when injected at suitable points to distribute it throughout entire body of water in process of purification. Activated carbon was injected into pipe lines carrying untreated influent; into water passing from clarifiers to settling basins; and into settled water ahead of filters. Tastes and odors were removed over the greater part of eight months by using 14 pounds per m.g. of Nuchar No. 2 for three months. Later, due to change in character of the water carbon treatment was gradually increased to 28 pounds per m.g. and held close to this point. Industrial wastes, algae, and stagnation all contribute to problem. Four grades of carbon, Nuchar No. 2, Nuchar No. 00 and Nuchar No. 000, manufactured by Industrial Chemical Sales Corp., and ranging in price from 5 to 10 cents per pound, have been used. A cheaper grade of carbon gave results equally as good. Carbex "A," manufactured by the American Active Carbon Co., was successfully tried. A disadvantage in use of powdered activated carbon is, that once successful treatment has been

established, one hesitates to reduce amount applied. Just how great must be factor of safety to insure continuous supply of palatable water from raw water which varies as does Saginaw supply, is questionable. Additional cost of a few dollars per million gallons is warranted, to help establish confidence of public in their water supply.—*R. E. Noble.*

Keeping Rapid Sand Filter Beds in Good Condition. JOHN R. BAYLIS. Water Works and Sewerage, 79: 124–6, 1932. Poorly designed underdrains and improper washing will cause beds to clog. Types of water which produce greatest clogging are those which cause gelatinous coating to form around sand grains and those high in organic matter. Waters of high organic content which contain manganese cause trouble in filter beds which is difficult to remedy. Fairly fine sand will also clog filters, or form hard spots in the bed which are not broken up by washing. The masses of clogged material or mud balls, may settle to the gravel surface, may be distributed throughout the sand, or may remain on the surface, depending on the amount of sand in the mud ball. Most satisfactory way to get rid of clogged places is to dig them out. Most clogging is along side-walls. The sand at these places should frequently be dug up. A hose, or rake, may sometimes be used to break up clogged places. Caustic soda has been used to limited extent for removing coating from sand grains.—*C. C. Ruchhoft (Courtesy Chem Abst.).*

Improved Mechanical Treatment of Water for Filtration. MARSDEN C. SMITH. Water Works and Sewerage, 79: 103–6, 1932. Three considerations control proper treatment of raw water after floc has been produced; (1) time that flow is kept in suspension, (2) mechanical treatment of floc while held in suspension, and (3) the proper return to incoming treated water of portion of settled floc and turbidity. A circulatory, twisting motion for a sufficient length of time will produce floc that will readily precipitate. If portion of floc is returned and mixed with incoming treated water, minimum amount of coagulant can be used. Paddles were used for mechanical stirring with a noticeable difference in settlement of floc, most of which settled within 50 feet of the paddles, while formerly, without stirring, floc settled along entire length of 700-foot tank.—*C. C. Ruchhoft (Courtesy Chem. Abst.).*

Aëration and Mixing Device. RICHARD F. WAGNER. Water Works and Sewerage, 79: 141–4, 1932. An "Air-O-Mix" has been put into use at Lynchburg, Va. This device thoroughly mixes air, water, and chemicals. It can be located at top of vertical coagulating tower, to receive all raw water, for aëration and improved coagulant mixing. Water level in coagulating tower could be maintained as heretofore. It can operate the year around without supervision, removing tastes, odor, and CO_2, and at less cost than by former method.—*C. C. Ruchhoft (Courtesy Chem. Abst.).*

Rainfall. ANON. Water and Water Engineering, 34: 400, 53–55, February 20, 1932; 401, 118–121, March 21, 1932 and 402, 153–155, April 20, 1932. Standard instruments are essential for accurate rainfall records. Details of standard rain gauge and of standard measure for daily records are described, as well

as Bradford, Seathwaite, and Octapent gauges for weekly or monthly readings, and defects of older forms of gauges are pointed out. Second part deals with rainfall computations, e.g., determination of average annual rainfall, period to be adopted, determination of mean rainfall over an area for various long periods and for few days. Effective rainfall, i.e., rainfall less evaporation and percolation loss, is used by water engineers in impounding schemes and is measured by weir gauges or by indirect means. Part three deals with exposure of a rain gauge, including over-shelter, over-exposure, and insplashing into the gauge. Standard height is that rim of gauge should be one foot above ground.—*W. G. Carey.*

Some Notes on Water Analysis. G. G. GEMMELL and R. G. THIN. Water and Water Engineering, **34**: 400, 64–67, February 20, 1932. FRESENIUS's and other methods of allocating mineral constituents found by analysis are criticised as incomplete and not always giving an adequate representation of probable composition, or properties, of water. Modern view is that in water ionization is complete, so that it is the possible composition of residue on evaporation which actually is calculated. Experiments on separation of some constituents, e.g. calcium carbonate, by fractional precipitation on boiling, were made and such separation is useful check. Precipitate is mainly calcium carbonate, partly owing to solubility of magnesium carbonate and partly because of double decomposition between other calcium salts and magnesium carbonate. No distinction is made between carbonate and bicarbonate in calculations, so that two waters with entirely different properties might be shown as having same analysis; examples of this are given. Measurement of pH value pictures conditions prevailing in water more accurately and change of pH after boiling may be of great importance, as is exemplified by various solutions. Large alteration of pH value, due to presence of very small quantities of silicates in water, is described and examples are given.—*W. G. Carey.*

Bakewell, England, Water Supply. T. MARSLAND. Water and Water Engineering, **34**: 400, 59–63, February 20, 1932. New supply is taken from spring and, after impounding, conveyed in cast iron pipes to new reservoir. Water, of highest degree of bacterial purity (analysis given showed one colony visible on agar after 24 hours at 37° and no B. Coli in 150 cc.), is very soft (40 p.p.m.) and has pH value 4.6. Sodium silicate and soda ash (thirty parts of each per million) are added under Venturi tube control. Two float columns, one connected with upstream section and one with throat section of Venturi tube, contain copper floats operating differential gear, which in turn controls vulcanite conical valves in chemical tanks, thus liberating chemicals.—*W. G. Carey.*

Recent Water Extensions in Newbury, England. N. S. BOWES. Water and Water Engineering, **34**: 400, 56–57, February 20, 1932. Good water is obtained from boreholes sunk 450 feet into chalk; yield from one of them was increased by 40 percent by exploding charge therein. Reservoir is fitted with "Pillinger-Bruston" pneumatic apparatus which propels water to buildings above level of supply. Air-pressure cylinders are partially filled with water which is driven

out as required by the air pressure to high points in neighbourhood and replaced by means of small automatically-controlled water pumps, thus keeping the air at the requisite pressure.—*W. G. Carey.*

Influence of Brine on the Filtration Coefficient through Sand. Anon. Water and Water Engineering, 34: 400,50, February 20, 1932. French scientist concludes that sand firmly retains salt from saline water and that increase in quantity of salt solution diminishes filtration coefficient. Sand through which saline water circulates will have reduced capacity for filtration even for pure water, this reduction being due not so much to viscosity of brine as to reduced permeability of sand due to salt content.—*W. G. Carey.*

Improvements in Dessau, Germany, Waterworks. T. Overhoff. Gas- und Wasserfach, 74: 45, 1025–1030, November 7, 1931. Town of 83,000 inhabitants has ground water supply containing 4 p.p.m. iron, 1.2 p.p.m. manganese, 30 to 40 p.p.m. free carbon dioxide, and 161 p.p.m. hardness. Iron precipitates on standing and water is plumbo-solvent. Prior to 1930, treatment was aëration, slow filtration, and treatment with alkali. New plant has been constructed, including totally submerged, water-lubricated, centrifugal pumps, motor-driven from Diesel plant. Automatic lime addition is regulated by pressure device in accordance with water flow, and is air-agitated. After liming and settlement for from 1 to 2 hours, water is aërated and filtered through rapid gravel filters, situated above old filters now used as storage. Wash water for filters represents from 2 to 3 percent of total output. Iron is reduced to from 0.8 to 1.2 p.p.m.; free carbon dioxide to 10 p.p.m.; manganese is entirely eliminated; and hardness raised by, at most, 27 p.p.m. calcium carbonate. Treated water is crystal clear and colorless.—*W. G. Carey.*

Water Supply Exhibits at German Building Exhibition, Berlin 1931. A Violet. Gas- und Wasserfach, 74: 40, 925–929, October 3, 1931. Berlin waterworks exhibited diagrams showing corrosion in hot water plants resulting from decomposition of bicarbonates by heat and effects of carbon dioxide and oxygen on iron apparatus and pipes. Tables and charts of water-demand fluctuations at different seasons in different types of districts were exhibited. Large colored panorama of water cycle in nature, as applied to water supplies of Germany, also shown.—*W. G. Carey.*

Short Report on the Construction of Works in the Letzlinger Moor to Supply Ground Water to Magdeburg. Götsch. Gas- und Wasserfach, 74: 39, 909–910, September 26, 1931. After boring and pumping experiments, it was decided to bore 18 wells about 220 feet deep, in two groups, each with pumping plant. Iron removal is to be by spraying into chambers with settlement for one hour, after which, water will flow to six open sand filters and thence to reservoir where water from river water works will be mixed with it.—*W. G. Carey.*

The Relative Persistence of B. Coli and B. Aërogenes in Nature. Fred O. Tonney and Ralph E. Noble. Jour. Bact., 22: 433, 1931. To investigate predominating rôle of *B. aërogenes*, cultures were planted in decayed stumps

with and without admixture with fecal material. Place counts at frequent intervals were made until organisms disappeared. After about 60 days, increase in *B. aërogenes* was considerable, in *B. coli* only slight. This suggests that excessive concentrations of *B. aërogenes* give a distorted sanitary picture, with little evidence of fecal pollution.—*Edw. S. Hopkins.*

Elimination of Tastes and Odors of Industrial Origin from Public Water Supplies. MORTIMER M. GIBBONS. Ind. Eng. Chem., 24: 977–82, 1932; cf. *C. A.* 26: 2536. Detailed data are given on use of activated carbon in conjunction with pre-chlorination, or with permanganate for removal of tastes, or odors, from water at Rahway, N. J. Stream is polluted with wastes from dyeing, phenol, lacquer solvents, etc. Powdered material is used, in doses varying from 1 to 20 p.p.m. applied as usual practice to coagulated water.—*Edw. S. Hopkins (Courtesy Chem. Abst.).*

Water Treatment. T. R. CAMP. Pro. Sixth Conf. Mass. State Ass'n. Master Plumbers, February 24, 1932, page 119. Matter exists in water in a colloidal, dissolved, or suspended state. Largest colloidal particles are 1 micron in diameter; smallest are 1 millicron in diameter or about size of largest molecule of dissolved matter. Substances occurring in natural waters are tabulated. Characters of dissolved mineral and organic matters are described. Principles of operation of slow sand and rapid sand filters are described and illustrated. Purpose of aërating water is explained and two aëration plants are illustrated. Softening, recarbonation, and control of corrosive characteristics of water are briefly discussed.—*H. E. Babbitt.*

Pulling Well Screens Safely by the Sand-Joint Method. Anon. Johnson National Drillers' Journal, August–September, 1932, page 1. Sand-joint method is best, because pressure must be applied evenly to all parts of screen. Principle is to lower a pulling pipe into screen and fill space between pulling pipe and screen about two-thirds full of sand. Sand is held in screen by means of plug of sacking wired to bottom of pulling pipe. Lifting force at the start should be applied slowly and steadily to pulling pipe. Pulling pipe should be hollow and should have holes just above the plug: inside of pulling pipe also contains sand. If pulling pipe gets stuck, sand within it can be bailed out, so that sand between pipe and screen runs into pulling pipe and is also bailed out, thus loosening the pulling pipe. Experience has been widely successful. —*H. E. Babbitt.*

A Remarkable Blowing Well. K. M. BROWN. Johnson National Drillers' Journal, August–September, 1932, p. 3. Air escaping from well only 150 feet deep blew sand, water, and egg-size gravel 100 feet into the air for two hours.—*H. E. Babbitt.*

Shooting Wells with Nitroglycerine. C. O. RISON. Johnson National Drillers' Journal, August–September, 1932, p. 4. Continued from last month. While time-bomb has come into general use in detonation of torpedo, other methods of detonation are still in use. Time-bomb and its connections and

firing are described. Time-bomb is lowered into well *after* torpedo has been placed. Methods of firing with jack squibs and electric squibs are described. Nitroglycerine freezes at temperatures below 55 to 60 degrees F. As use of frozen explosives is dangerous, lower freezing compounds are sometimes used. Explosives can be thawed safely by placing in container and suspending in barrel of tepid water.—*H. E. Babbitt.*

Estimating Yield of Well from Drawdown. Discussion of earlier article. C. N. Ward. Johnson National Drillers' Journal, August–September, 1932, p. 5. Test data showing increasing specific yield with increasing drawdown should be seriously scrutinized. Theory predicts and under most conditions results have shown, smaller specific yields with increasing drawdown. However, case is illustrated in which the reverse may be true. Four conditions essential for successful well test are described. Elevation of water surface in well may not represent true free surface of water in surrounding strata. Method of measuring true level is given. Observations made during various tests are given to illustrate points made.—*H. E. Babbitt.*

Pump Operating on New Principle is Patented. Anon. Johnson National Drillers' Journal, August–September, 1932, p. 10. Torbio Bellocq, of Buenos Aires, has secured U. S. patent on deep-well pump which operates with no moving parts in well except a foot-valve at bottom of suction pipe. At upper end of this pipe a piston is made to reciprocate rapidly in short strokes, setting up alternate waves of compression and rarefaction. This rapid vibration of piston causes foot-valve to open and water flows from well.—*H. E. Babbitt.*

Dual Uses of Buttress Dams. C. V. Adams and F. Hudson, Jr. Civil Engineering, 2: 8, 472, August, 1932. Greater economy and greater safety are attained by utilizing space within buttress dams for housing of hydroelectric power, or water-purification, units, and by utilizing structural members of dam to form tanks and basins required for water purification plant. Theoretical, graphical comparison of buttress and gravity dams shows former to be the more economical in material. Curves comparing quantities of materials in various dams shows least amount in Rodriguez buttress dam. Water purification plant at Clinton, Okla., is housed in buttress dam forming impounding reservoir, structural members of dam also forming sides of storage basins. Saving enjoyed by this construction is shown graphically for five projects. Based on cost of gravity type dam, saving in buttress type dam amounts to from 30 to 40 percent. Increased structural safety is attained and loss of head between dam and various reservoirs is less.—*H. E. Babbitt.*

Deflections of Swiss Dams Measured. F. A. Noetzli. Civil Engineering, 2: 8, 489, August, 1932. Method involves triangulation with precision theodolites. Linear deflections of dams were determined by intersection, with accuracy between 0.01 and 0.02 inch. All dams investigated, both straight gravity and arch type, showed permanent deformations of considerable magnitude. On some dams there were indications of downstream movement continuous over several years. Vertical movements were practically independent

of water pressure and of effect of water-soaking on concrete, and seemed to result entirely from temperature changes. Deflections are a combination of elastic and permanent deformations. Amsteg dam deflected 0.14 inch when filled in 1922. This increased by about 0.12 inch up to 1928. It became evident that the whole dam was moving downstream by small but measurable amounts. Schraeh Dam, gravity dam, moved permanently downstream about 0.51 inch. Horizontal movements and dislocations in other dams are recorded. Downstream movement of arch dams may increase their safety by closing construction joints, but no possible benefit can be derived from such a movement. Desirability of building gravity dams curved upstream in plan, is indicated.—*H. E. Babbitt.*

Improving Fire Insurance Risks. C. M. ANDREWS. Civil Engineering, 2: 8, 506, August, 1932. Plan is proposed to secure funds for waterworks improvements from the savings in fire insurance premiums resulting from higher classification. Individual policies are to be placed in trust in bank which is to advance money for the improvement.

Hydraulic Flow Against Gradient. C. S. JARVIS. Civil Engineering, 2: 8, 508, August, 1932. Water can flow uphill either because of differences in specific gravity between fresh and salt water, or becase of momentum of moving water prism. Such phenomena have been observed at Army Point, Benicia Arsenal, Calif., and in Cape Cod Canal at Buzzards Bay, Mass.—*H. E. Babbitt.*

Construction Methods on the Cle Elum Dam. W. H. GARDINER. Civil Engineering, 2: 9, September, 1932. Difficulties of tunneling in soft ground and other problems successfully met. Dam now under construction in Yakima Valley in Washington, is an earth dam, which will raise water level 110 feet and impound 420,000 acre-feet. It contains 1,000,000 cubic yards of embankment; 21,000 cubic yards of tunnel excavation; and 33,000 cubic yards of concrete. Total cost is estimated as $1,750,000, plus $980,000 for rights of way, stripping, etc. Details of construction problems and their solution are given. Completion of spillway and main dam will be accomplished in 1933.—*H. E. Babbitt.*

Construction Equipment for Hoover Dam. NORMAN S. GALLISON. Civil Engineering, 2: 9, 573, September, 1932. Construction equipment had to be selected with following in view: (1) inaccessibility of site; (2) extreme climatic conditions; (3) large tonnages involved; and (4) short period allowed for completion. During 1931 temperature varied between 12° and 136°F. Because of immense amount of material involved, equipment had to be extremely rugged and relatively long-lived; while its ability to perform without replacement and according to a rigid time schedule, was of prime importance. Factors influencing selection were: reputation gained from experience; ability of manufacturers to meet unusual specifications; and ruggedness. In special cases mobility and portability were considered. Large trucks were required for disposal of 2,000,000 cubic yards of muck. Trucks have 9 to 14 cubic yards capacity and weigh 20 to 32 tons when loaded. Drilling in tunnels was accomplished by

mobile "jumbos." In larger tunnels, mucking was done directly into trucks by steam shovels. Twenty-seven miles of railroad have been constructed for moving 27 million tons of aggregate and 5 million tons of other material. Concrete mixing plant is expected to produce 170,000 cubic yards per month. Some additional large items called for include 300,000 cubic yards of rock excavation for bases of intake towers; 1,000,000 cubic yards of earth fill for coffer dams; 150,000 cubic yards of loose rock stripped from canyon walls; 35,000,000 pounds of reinforcing steel to be placed; besides other important items of great magnitude.—*H. E. Babbitt.*

Uplift Pressure in Masonry Dams. I. E. Houk. Civil Engineering, 2: 9, 578, September, 1932. Recorded measurements summarized and new theory developed. In design of dam, engineer needs to know: first, intensity of uplift at different locations between upstream and downstream faces of dam; and second, proportion of horizontal cross-sectional area on which this pressure acts. Diagram shows maximum uplift pressures recorded at ten dams. No satisfactory data applicable to second question are available. For purposes of design, it seems safe to conclude that assumptions of uplift pressure applicable to foundation level of masonry dam will be more than ample for horizontal construction joints at different elevations above the base.—*H. E. Babbitt.*

Effect of Uplift on Stability of Straight Gravity Dams. D. C. Henry. Civil Engineering, 2: 9, 580, September, 1932. Uplift is ordinarily less than 50 percent of full reservoir head at 10 percent of base length from water face and diminishes to zero at toe. Proportion of effective uplift area to total area of any given horizontal section is not known. Area along a concrete construction joint approaches the pore area. Potential relative area is likely to decrease as vertical loading increases. Sliding factor for judging safety of dam against downstream movement is not a logical criterion except in case of extended horizontal stratification planes in foundation rock. Shear-friction factor is more logical guide than sliding factor. The Levy requirement is ordinarily excessive for dams lower than 500 feet high.—*H. E. Babbitt.*

Arc-Welded Field Joints in Large Size Pipe. Contract. Engrs. Monthly, 25: 2, 38, August, 1932. Application of electric arc welding process to joining of sections of large oil and gas transmission lines is not new; but its use on pipe over 30 inches in diameter has been very rare. Shielded arc process has been adopted for 56-inch water supply line now being completed on Hetch Hetchy Aqueduct in San Joaquin Valley. Pipe is electrically welded with Lincoln automatic arc welding equipment and, in the field, several sections are fused ingo single unit with the shielded arc. Pipe shell varies in thickness from $\frac{5}{16}$ to $\frac{1}{2}$-inch. Current is supplied by 8 gasoline-engine-driven portable welding units of type used on petroleum pipe line welding. Trench is dug before lining up of pipe and pipe is tack-welded in five or six places and rolled into place in sections wherever terrain permits. Welding is done in two shifts of 8 hours each. Day crew works on outside of pipe and night crew follows on inside.—*Geo. C. Bunker.*

Riverside Develops New Water Supply Source. J. F. DAVIDSON. Western City, 8: 8, August, 1932. A 20-inch well, 353 feet deep, was drilled on new water-bearing property purchased by city in order to develop supplementary source of supply. Well water is raised by deep-well pump with capacity of 2250 g.p.m. into small baffled sand-arresting settling basin, from which it is pumped through 12,000 feet of 24-inch Hume centrifugal concrete pipe to distribution system. Pipe was manufactured in 30-foot lengths, with bell and spigot joints, and calked with dry cement, having just enough moisture to hold it in place. LARNER-JOHNSON check valve was installed on discharge side of booster pump to protect both it and the motor against reverse rotation when power interruption occurs. LARNER-JOHNSON surge-suppressor was installed in by-pass, to eliminate water-hammer in pump discharge line on shut-downs. —*Geo. C. Bunker.*

Los Angeles Method of Well Testing. S. M. DUNN. West Constr. News & High. Bldr., 7: 13, 383, July 10, 1932. To secure portable source of electric power, of capacity greater than that of usual gasoline-engine unit, for use in emergencies, Los Angeles Department of Water and Power purchased a LeRoi-Westinghouse engine-generator, capable of driving motors up to 150 h.p. This equipment has been used in operating pump to determine relation between pumping level and capacity in seven new wells.—*Geo. C. Bunker.*

Southern California Should Soften Water. CHESTER S. SMITH. West. Constr. News & High. Bldr., 7: 16, August 25, 1932. Reason that California has only one municipal water-softening plant out of about 125 such plants in U. S. is that in the other cities majority of citizens have not been sold on the idea. When public has been educated to fact that three or four cents additional per 1000 gallons in water bills will be more than refunded by amount saved in soap, soap flakes, cleansers, face and hand lotions, plumber's bills, gas or electric-heater bills, etc., then water-softening plants will be demanded. Construction costs of such plants range from \$35,000 to \$50,000 per m.g.d. capacity. In table are given operation costs in 24 cities, ranging from minimum of \$0.0022 to maximum of \$0.0600 per 1000 gallons. Soft water costs nothing, when costs of softening are balanced against savings to each individual.—*Geo. C. Bunker.*

Note on Stopping Leaks in A Dam by Grouting Methods. M. AUBERTIN. Annales des Ponts et Chaussées, 1932: 3, 417–434, May–June, 1932. Ban de Champagney dam, 785 meters [2575 feet] long and with maximum height of 36 meters [118 feet] above ground level, across Rahin River, Haute-Sâone, France, was completed in 1926, and impounds 13,000,000 cubic meters [10,600 acre-feet]. It is founded on marl and red sandstone. When it was attempted to fill reservoir, serious leaks developed, both through masonry and through underlying strata. Exploratory borings showed that foundation, in particular, was very pervious. It was therefore decided to grout both defective masonry and foundations by FRANÇOIS process, in which "silicazation" precedes application of cement grout, to stabilize particles forming porous rock, and so prevent their displacement when subjected later to pressure built up by grout-

ing. This is accomplished by injecting solutions of sodium silicate and aluminum sulphate, either simultaneously or consecutively. Holes numbering 144 and totalling 3512 meters [11,522 feet] were drilled; 19 metric tons of salts were used in silicazation and 5790 metric tons of Portland cement as grout. Results were very satisfactory, practically all leaks disappearing. Cost of operation was 3,850,000 francs plus 55,000 Reichmarks ($167,000).—*R. DeL. French.*

On the Detection of Minute Quantities of Carbon in Water. M. PICON. Jour. de Pharmacie et de Chimie, 16: 5–20, July 1, 1932. Carbon content in water is of great interest, as among most reliable chemical indications of organic pollution. Until recently, methods of detecting small quantities of carbon were defective and unreliable, but now at least three methods are available, those of FRIEDMANN and KENDALL, of SIMON, and of NICLOUX. Modification of last-mentioned, especially designed for use in examination of waters for public or industrial purposes, is advocated and fully described. Tables are also included showing carbon content of various waters of France, both surface and subterranean, and showing effect on results of analysis of minor variations in procedure. Comparisons with potassium permanganate method are also given.—*R. DeL. French.*

The Application of Electric Prospecting to the Study of Tunnel and Dam Projects. C. SCHLUMBERGER and E. G. LÉONARDON. Annales des Ponts et Chaussées, 1932: 2, 271–289, March–April, 1932. Method described is based upon measurements of specific resistivity, which is very low for dense and nonporous rocks, and higher for those which are lighter and partly saturated with water, with, or without, presence of metallic salts. Three examples of application of method are given: (1) at Bridge River tunnel of British Columbia Electric Railway Company, (2) at site of proposed Ogden Island dam of St. Lawrence Waterways, and (3) at Masson tunnel of MacLaren power development on Lièvre River; all in Canada. Authors conclude that method is rapid, accurate, and economical, and particularly adapted to reconnaissance and to indicating where accurate soundings should be made, as it gives only a general indication of character of underlying rock, and not precise data required for detail design of structures.—*R. DeL. French.*

The Electrical Purification of Water. J. BILLITER. Transactions Electrochem. Soc., 50: 1931. Large-scale electrical purification of water in three-compartment cells is not satisfactory on account of high cost of diaphragms. Author has devised two-compartment cell for direct electrolysis of water with feeble electro-osmotic effect. Water containing 150 to 600 mg. per litre of salts, or even more, can be easily treated, giving product containing 6 to 9 mg. per litre of salts. New cell has two compartments, one of a clay composition and the other of asbestos.—*R. DeL. French.*

The Sterilization of Water by Silver. SARROT DU BELLAY. La Nature, No. 2887, 149–150, August 15, 1932. Katadyn process of water sterilization, invented by Dr. KRAUSE of Munich, uses very finely divided silver as filtering

medium, some of which, but not more than 0.00006 grm. per liter, dissolves as silver ion in combination with dissolved oxygen of water, and creates electrical field fatal to bacteria and to some moulds, and also, perhaps, to ferments. Small quantity of silver in solution is not at all injurious to health. Water sterilized by this process has power to limited extent, of sterilizing additional raw water; process is therefore useful in maintaining swimming pools. No preliminary treatment of raw water is required; equipment is cheap and simple and therefore adapted to small supplies.—*R. DeL. French.*

Action on Microbes of Metals at a Distance. G. A. Nodson and C. A. Stern. Revue Scientifique, No. 15, 476, August 13, 1932. Previous series of experiments by these authors dealt with effects of aluminum, copper, and lead; present series deals with nickel, silver, gold, and platinum. Sterilizing effect of metals varies with their atomic weights. Metallic salts are less active than pure metals, but their influence is felt at greater distance. Action is greatly decreased by interposition of quartz, or paper, shield between sterilizing agent and bacteria. It is thought that, under influence of radio-activity of surrounding medium, secondary radiation of considerable germicidal power, like X-rays, takes place from surface of metal, and that colonies of bacteria are bombarded by electrons, with fatal effect.—*R. DeL. French.*

Die chemische Zellatmung als Hilfsmittel der bakteriologischen Trinkwasseruntersuchung. (The Chemical Respiration of the Cell as an Aid in Bacteriological Examination of Drinking Water.) W. Liese. Zentralbl. für Bakt., Parasit., und Infekt., I Abt., 124: 560–568, 1932. In actively growing cultures of *B. coli* and other intestinal bacteria, colorless nitro-anthraquinone is reduced to red amino-anthraquinone; nor is color change subject, as in case of methylene blue, to interference from atmospheric oxygen. To 1 cc. of ordinary bouillon of pH 7.5 (buffered, if needful) add 1 cc. of water sample. Then (1) incubate at 37° for about 20 hours; (2) add 0.5 cc. of fresh aqueous solution (1:50) of nitro-anthraquinone; (3) set in water-bath at 40° for 4 hours; and (4) read off degree of color developed. In last-mentioned operation, four degrees only of color are recognized: colorless; barely perceptible tinge of delicate yellowish red; distinct delicate red; and red to deep dark red; which, for convenience, are denoted respectively as 0, x, xx, and xxx. Ten such tubes are taken for each test. Results are qualitative, rather than quantitative. Considerable number of examples from actual practice are given. No difficulty arises in interpreting results when either (1) all 10 tubes read 0; in which case water is perfectly safe; or (2) all, or majority, of tubes read xxx; in which case recent infection is indicated. Rules are given for interpretation of results intermediate between these extremes. The great advantage claimed is that reliable indications are obtained within 24 hours.—*Frank Hannan.*

NEW BOOKS

Proceedings of the Sixth Annual Conference, Maryland-Delaware Water and Sewerage Association, Cumberland, Maryland, April 12 and 13, 1932. 128 pages. **Elementary Sewage Disposal.** L. V. Carpenter. 5–14. **Discussion.** G. L. Hall. Tabulation of location, population served, and methods

of treatment at thirty-eight sewage treatment plants in Maryland. **Admission of Industrial Wastes into Sewerage Systems. Symposium. Control of the Pollution of Streams.** 35–63. **Procedure in Delaware.** R. C. BECKETT. Pollution of Brandywine Creek above Wilmington prompted a pollution survey of stream which disclosed its sources and resulted in installation of waste treatment plants in many industrial works. **Procedure in Pennsylvania.** Discussion of state laws, basic policies, stream studies, and of manner of coöperation with municipalities, industries, and nearby states. Sanitary Water Board under State Department of Health is administrative agency. Bureau of Engineering of that department is investigatory, recommending, and enforcement agent. **Procedure in West Virginia.** E. S. TISDALE. State stream pollution law provides that state agencies shall have control of public health, power, and fish and game matters. No new government agencies are established. Coöperation of municipalities in stream pollution control has been unsatisfactory, but work with industries has shown real progress. **Procedure in Maryland.** ABEL WOLMAN. Control of stream pollution is administered by State Department of Health. Other interested agencies refer to this department matters for action on pollution control. **The Relation of the Water Department to the Medical Health Department.** HARVEY H. WEISS. 64–68. Plea for closer coöperation between health officer and water department operating personnel. **Influence of Soils, Rocks and Minerals upon the Potability of Water.** J. J. SHANK. Surface waters acquire mineral constituents by flowing over soils and rocks of various solubilities. Ground waters which penetrate beneath upper crust of earth's surface acquire much greater mineral content than surface waters, due to longer and more intimate contact with soil and rock materials. Action taking place between ground waters and soil and rock materials is of nature of solution, substitution, addition, or combination. **Waste and Stored Water Supplies, with Particular Reference to Ashland, Pennsylvania, and Cumberland, Maryland.** H. E. BECKWITH. 78–86. Water waste survey at Ashland, Pennsylvania, resulted in reduction of consumption by about 40 percent which was in effect equivalent to increase in impounded storage of about 100 percent. At Cumberland, Maryland, reduction of waste amounting to 33 percent of total consumption is considered to be equivalent to increase in impounded storage of about 70 percent. **Developments on Powdered Activated Carbon in Water Plants During Recent Months.** F. E. STEWART. 87–95. In addition to its first developed use for removal of tastes and odors, activated carbon is effective for elimination of putrefaction and liquefaction of settling basin sludge; for improvement in coagulation, with consequent reduction of dosage of coagulant; for adsorption of organic matter, thus controlling fluctuations in chlorine demand; and for increasing chlorine residual by application ahead of prechlorination dosage. Bibliography of 10 references appended. **Discussion.** C. A. HECHMER. Experience at Washington Suburban Sanitary District has shown that best results are obtained when carbon is applied to raw water with coagulant. It is fed by dry-feed machines at rate of 0.1 grain per gallon at cost of 85 cents per million gallons. **The New Thomas W. Koon Dam.** H. H. ALLEN. 96–111. New dam across Evitts Creek 12 miles above Cumberland, Maryland, is of solid masonry overflow type and creates reservoir 2.2 miles long covering 268

acres and impounding 2300 m.g. Four classes of concrete were used in construction. Control of concrete proportioning and handling involved employment of concrete technician, field laboratories, new device for determination of surface moisture, proportioning of all ingredients by weight, graphical recording of all factors, use of large size aggregate, use of buckets for transportation of concrete, and use of electric vibrating puddlers. **Question Box.** 112–121. **Reservoir Treatment with Copper Sulphate.** At Baltimore, 0.5 p.p.m. applied weekly will retard algae growth below point causing taste. At Hyattsville, Maryland, 0.35 p.p.m. applied semi-weekly found effective and more satisfactory than prechlorination. At Harrisburg, Pennsylvania, usual recommendation is 2 pounds copper sulphate per million gallons; some destruction of small trout, but of no other fish, has been noted. At Cumberland, Maryland, 4 pounds per million gallons is applied without any undue killing of fish. **Charge for Water Used by Gas Refrigerators.** Answers indicate that gas refrigerators usually require from 3 to 5 gallons per hour. Ordinary small meters will not register this amount. No well-settled policy for charging for this water was indicated.—*R. L. McNamee.*

Effects of the Irpino Earthquake of 23 July, 1930 on the Works of the Apulian Aqueduct, South Italy. Dr. Ing. Pietro Celentani-Ungaro. Separate of 31 pp., 12 x 8½ inches, abundantly illustrated, with views, maps, plans, and graphs; from L'Ingegnere, 5: no. 10, October 1931. Great importance attaches to this exhaustive portrayal. For (*a*) aqueduct system is located in difficult terrain in one of world's best known earthquake regions; (b) undertaking had, for this reason, from its inception, to withstand force of much weighty adverse criticism; (*c*) shock of 23 July, 1930 was exceptionally violent and large sections of system, including its most important works, was severely shaken, being within the VI isoseismic (Moncalli) and considerable portion even within VII (epicentric, or maximum shock intensity, region being that within X); (*d*) notwithstanding which, no interruption of flow in main aqueduct occurred; while in primary branch, to Province of Foggia, in which flow ceased for a time, conditions were restored to normal within 13 or 14 hours and reservoirs took ample care of all demands during interruption; and (*e*) every part of system within affected region has undergone minute inspection since the shock and damage revealed is reported in great detail, with discussion of stresses shown to have developed and of how these may best be countered. Flow in main aqueduct is about 120 m.g.d., supplying a scattered population of 2,500,000. Construction is chiefly masonry, either concrete, or brick and mortar. Flow is chiefly gravity and contour of section varies with terrain; dimensions where approximately rectangular are 6½ feet wide by 9 feet high. Length in affected region is about 53 miles, including 45 of tunnel, 7 in trench, and about 1 mile carried on bridges. In same region there are in branches, etc. about 18 miles of gravity and 105 miles of pressure conduit, latter being constructed severally of steel, cast iron, eternit (cement-asbestos), and reinforced concrete. Advantage has been taken of the plentiful material to amass a collection, probably unrivalled, of data on earthquake effects on aqueducts. Comparisons are made with other similar disasters in Italy and with Tokio and San Francisco. Useful constructional hints are given by which earthquake

damage may be guarded against and others, perhaps more useful still, on how to be prepared to repair at short notice damage which does occur. It was noticed that supply from Caposele springs from which aqueduct is fed increased suddenly after the shock by about 4 percent, but gradually fell back, in course of a month or so, to former level. The courage and skill, with which this great undertaking, of which Italy may well feel proud, has been carried out, have been more than amply vindicated.—*Frank Hannan.*

Die chemische Untersuchung von Wasser und Abwasser. (The Chemical Examination of Water and Sewage.) J. Tillmans. Second Edition; with 28 illustrations. 1932. Wilhelm Knapp: Halle (Saale). Vol. 17 of the Laboratoriumsbücher für die chemische und verwandte Industrien. New and thoroughly revised edition of the well-known text-book of Tillmans. Most notable additions are in field of attack on iron by carbon dioxide and oxygen, which author has made peculiarly his own, and in field of sewage examination, where the numerous valuable improvements due to Dr. Bach of the Emschergenossenschaft receive recognition.—*Frank Hannan.*